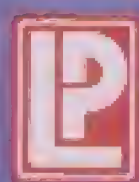


# SOIL MECHANICS AND FOUNDATIONS

16TH EDITION



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# Contents

## **PART I ELEMENTARY PROPERTIES**

### **CHAPTER 1 INTRODUCTION**

1.1. SOIL AND SOIL ENGINEERING	1
1.2. HISTORY OF DEVELOPMENT OF SOIL MECHANICS	4
1.3. FIELD OF SOIL MECHANICS	6
1.4. SI UNITS	8

### **CHAPTER 2 PRELIMINARY DEFINITIONS AND RELATIONSHIPS**

2.1. SOIL AS A THREE PHASE SYSTEM	9
2.2. WATER CONTENT, DENSITY AND UNIT WEIGHTS	10
2.3. SPECIFIC GRAVITY	12
2.4. VOIDS RATIO, POROSITY AND DEGREE OF SATURATION	13
2.5. DENSITY INDEX AND RELATIVE COMPACTION	15
2.6. FUNCTIONAL RELATIONSHIPS	17
2.7. PACKING OF UNIFORM SPHERES	28
2.8. EXAMPLES FROM COMPETITIVE EXAMINATIONS	30

### **CHAPTER 3 DETERMINATION OF INDEX PROPERTIES**

3.1. GENERAL	37
3.2. WATER CONTENT	37
3.3. SPECIFIC GRAVITY	41
3.4. PARTICLE-SIZE DISTRIBUTION	44
3.5. SIEVE ANALYSIS	44
3.6. SEDIMENTATION ANALYSIS : THEORY	46
3.7. PIPETTE METHOD	50
3.8. HYDROMETER METHOD	52
3.9. PARTICLE SIZE DISTRIBUTION CURVE	57
3.10. CONSISTENCY OF SOILS	59
3.11. DETERMINATION OF LIQUID AND PLASTIC LIMITS	62
3.12. SHRINKAGE LIMIT	66
3.13. DETERMINATION OF IN-SITU DENSITY, VOIDS RATIO AND DENSITY INDEX	73
3.14. ACTIVITY OF CLAYS	76
3.15. SENSITIVITY OF CLAYS	77
3.16. THIXOTROPY OF CLAYS	78
3.17. COLLAPSIBLE SOILS	78
3.18. AVERAGE DIAMETER OF A GROUP OF PARTICLES	81
3.19. SPECIFIC SURFACE	81
3.20. EXAMPLES FROM COMPETITIVE EXAMINATIONS	85
3.21. LABORATORY EXPERIMENTS	87

### **CHAPTER 4 CLASSIFICATION OF SOILS**

4.1. GENERAL	111
4.2. PARTICLE SIZE CLASSIFICATION	113



4.3.	TEXTURAL CLASSIFICATION	111
4.4.	HIGHWAY RESEARCH BOARD (HRB) CLASSIFICATION	113
4.5.	UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)	115
4.6.	INDIAN STANDARD CLASSIFICATION SYSTEM, ISCS (IS : 1498-1970)	120
4.7.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	132
<b>CHAPTER</b>	<b>5 SOIL STRUCTURE AND CLAY MINERALOGY</b>	
5.1.	SOIL STRUCTURE	133
5.2.	SOLID PARTICLES IN SOILS	134
5.3.	ATOMIC AND MOLECULAR BONDS	134
5.4.	INTERPARTICLE FORCES IN A SOIL MASS	139
5.5.	SINGLE GRAINED STRUCTURE	140
5.6.	HONEYCOMB STRUCTURE	141
5.7.	FLOCCULENT AND DISPERSED STRUCTURES	141
5.8.	STRUCTURE OF COMPOSITE SOILS	142
5.9.	CLAY MINERALS	142

## PART II SOIL HYDRAULICS

<b>CHAPTER</b>	<b>6 SOIL WATER : EFFECTIVE AND NEUTRAL STRESSES</b>	
6.1.	MODES OF OCCURRENCE OF WATER IN SOIL	147
6.2.	ADSORBED WATER	149
6.3.	CAPILLARY WATER	150
6.4.	CAPILLARY TENSION, CAPILLARY POTENTIAL AND SOIL SUCTION	153
6.5.	CAPILLARY PHENOMENON IN SOILS : CAPILLARY ZONES	156
6.6.	SHRINKAGE AND SWELLING OF SOILS	157
6.7.	SLAKING OF CLAY	158
6.8.	BULKING OF SAND	159
6.9.	FROST ACTION	159
6.10.	STRESS CONDITIONS IN SOIL : EFFECTIVE AND NEUTRAL PRESSURES	163
6.11.	CAPILLARY SIPHONING	168
6.12.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	169
<b>CHAPTER</b>	<b>7 PERMEABILITY</b>	
7.1.	INTRODUCTION	177
7.2.	DARCY'S LAW	177
7.3.	DISCHARGE VELOCITY AND SEEPAGE VELOCITY	179
7.4.	VALIDITY OF DARCY'S LAW	179
7.5.	POISEUILLE'S LAW OF FLOW THROUGH CAPILLARY TUBE*	180
7.6.	FACTORS AFFECTING PERMEABILITY	183
7.7.	COEFFICIENT OF ABSOLUTE PERMEABILITY	185
7.8.	DETERMINATION OF COEFFICIENT OF PERMEABILITY	186
7.9.	CONSTANT HEAD PERMEABILITY TEST	188
7.10.	FALLING HEAD PERMEABILITY TEST	189
7.11.	THE JODHPUR PERMEAMETER	189
7.12.	THE CAPILLARITY-PERMEABILITY TEST	194
7.13.	PERMEABILITY OF STRATIFIED SOIL DEPOSITS	195



7.14.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	198
7.15.	LABORATORY EXPERIMENTS	201
<b>CHAPTER 8 WELL HYDRAULICS</b>		
8.1.	INTRODUCTION	205
8.2.	SOME DEFINITIONS	205
8.3.	STEADY RADIAL FLOW TO A WELL : DUPUIT'S THEORY	207
8.4.	FIELD DETERMINATION OF $k$ AND $T$ : PUMPING OUT TESTS	210
8.5.	PUMPING IN TESTS	213
8.6.	INTERFERENCE AMONG WELLS	214
8.7.	FULLY PENETRATING ARTESIAN GRAVITY WELL*	215
8.8.	PARTIALLY PENETRATING ARTESIAN WELL*	215
8.9.	SPHERICAL FLOW IN A WELL*	216
8.10.	FLOW TOWARDS OPEN WELL : RECUPERATION TEST	217
8.11.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	221
<b>CHAPTER 9 SEEPAGE ANALYSIS</b>		
9.1.	HEAD, GRADIENT AND POTENTIAL	223
9.2.	SEEPAGE PRESSURE	224
9.3.	UPWARD FLOW : QUICK CONDITION	225
9.4.	TWO DIMENSIONAL FLOW : LAPLACE EQUATION	230
9.5.	GRAPHICAL METHOD OF FLOW NET CONSTRUCTION	232
9.6.	FLOW NET BY ELECTRICAL ANALOGY	233
9.7.	APPLICATIONS OF FLOW NET	234
9.8.	SEEPAGE THROUGH ANISOTROPIC SOIL	236
9.9.	DEFLECTION OF FLOW LINES AT INTERFACE OF DISSIMILAR SOILS	238
9.10.	PHREATIC LINE OF AN EARTH DAM	238
9.11.	PHREATIC LINE IN AN EARTH DAM WITH NO FILTER	240
9.12.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	244
<b>CHAPTER 10 SEEPAGE BELOW HYDRAULIC STRUCTURES</b>		
10.1.	INTRODUCTION	249
10.2.	PIPING : EXIT GRADIENT	250
10.3.	KHOSLA'S THEORY	250
10.4.	COMPOSITE PROFILES : SCHWARZ-CHRISTOFFEL TRANSFORMATION	252
10.5.	PAVLOVSKY'S METHOD : FINITE DEPTH PROBLEMS	261
<b>CHAPTER 11 DRAINAGE AND DEWATERING</b>		
11.1.	INTRODUCTION	267
11.2.	DITCHES AND SUMPS	267
11.3.	WELL POINT SYSTEM	267
11.4.	SHALLOW WELL SYSTEM	269
11.5.	DEEP WELL DRAINAGE	269
11.6.	VACUUM METHOD : FORCED FLOW	270
11.7.	ELECTRO-OSMOSIS METHOD	271
11.8.	SEEPAGE ANALYSIS	272
11.9.	PROTECTIVE FILTERS	275



## PART III

### ELASTICITY APPLIED TO SOILS

#### CHAPTER 12 ELEMENTS OF ELASTICITY\*

12.1.	STATE OF STRESS AT A POINT	279
12.2.	EQUILIBRIUM EQUATIONS	280
12.3.	EQUILIBRIUM EQUATIONS FOR SATURATED SOIL BODY	282
12.4.	STRAIN COMPONENTS : STRAIN TENSOR	284
12.5.	COMPATIBILITY EQUATIONS	285
12.6.	BOUNDARY CONDITION EQUATIONS	286
12.7.	GENERALISED HOOKE'S LAW : HOMOGENEITY AND ISOTROPY	287
12.8.	TYPICAL VALUES OF MODULUS OF ELASTICITY AND POISSON'S RATIO	288
12.9.	TWO DIMENSIONAL PROBLEMS	288
12.10.	COMPATIBILITY EQUATION IN TWO DIMENSIONAL CASE	289
12.11.	STRESS FUNCTION	291
12.12.	EQUILIBRIUM EQUATIONS IN POLAR CO-ORDINATES	292
12.13.	COMPATIBILITY EQUATIONS AND STRESS FUNCTION IN POLAR COORDINATES	293
12.14.	CYLINDRICAL CO-ORDINATES	294

#### CHAPTER 13 STRESS DISTRIBUTION : I

13.1.	INTRODUCTION : STRESSES DUE TO SELF WEIGHT	295
13.2.	CONCENTRATED FORCE : BOUSSINESQ EQUATIONS	296
13.3.	PRESSURE DISTRIBUTION DIAGRAMS	299
13.4.	VERTICAL PRESSURE UNDER A UNIFORMLY LOADED CIRCULAR AREA	303
13.5.	VERTICAL PRESSURE DUE TO A LINE LOAD	305
13.6.	VERTICAL PRESSURE UNDER STRIP LOAD	306
13.7.	VERTICAL PRESSURE UNDER A UNIFORMLY LOADED RECTANGULAR AREA	307
13.8.	EQUIVALENT POINT LOAD METHOD	311
13.9.	NEWMARK'S INFLUENCE CHART	312
13.10.	WESTERGAARD'S ANALYSIS	316
13.11.	CONTACT PRESSURE	322
13.12.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	323

#### CHAPTER 14 STRESS DISTRIBUTION : II\*

14.1.	VERTICAL LINE LOAD	325
14.2.	HORIZONTAL LINE LOAD	327
14.3.	UNIFORM VERTICAL LOAD OVER A STRIP	328
14.4.	CONCENTRATED FORCE : BOUSSINESQ PROBLEM	329
14.5.	NORMAL LOAD OVER A CIRCULAR AREA	331
14.6.	TRIANGULAR AND OTHER LOADINGS	332
14.7.	PRINCIPAL STRESSES AND MAXIMUM SHEAR	335

## PART IV

### COMPRESSIBILITY

#### CHAPTER 15 ONE DIMENSIONAL CONSOLIDATION

15.1.	INTRODUCTION	339
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15.2.	<u>THE CONSOLIDATION PROCESS : SPRING ANALOGY</u>	340
15.3.	<u>CONSOLIDATION OF Laterally CONFINED SOIL</u>	341
15.4.	<u>CONSOLIDATION OF UNDISTURBED SPECIMEN</u>	345
15.5.	<u>TERZAGHI'S THEORY OF ONE-DIMENSIONAL CONSOLIDATION</u>	346
15.6.	<u>SOLUTION OF THE CONSOLIDATION EQUATION*</u>	348
15.7.	LABORATORY CONSOLIDATION TEST	354
15.8.	CALCULATION OF VOIDS RATIO AND COEFFICIENT OF VOLUME CHANGE	355
15.9.	<u>DETERMINATION OF COEFFICIENT OF CONSOLIDATION</u>	357
15.10.	<u>SECONDARY CONSOLIDATION</u>	371
15.11.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	372
15.12.	LABORATORY EXPERIMENTS	389
<b>CHAPTER</b>	<b><u>16 THREE DIMENSIONAL CONSOLIDATION*</u></b>	
16.1.	<u>INTRODUCTION</u>	395
16.2.	<u>THREE DIMENSIONAL CONSOLIDATION EQUATION</u>	395
16.3.	CONSOLIDATION EQUATION IN POLAR COORDINATES FOR AXIS SYMMETRIC CASE	397
16.4.	SOLUTION OF THREE DIMENSIONAL CONSOLIDATION EQUATION	398
16.5.	CONSTRUCTION OF VERTICAL SAND DRAINS	401
16.6.	EFFECT OF PERIPHERAL SMEAR	405
<b>CHAPTER</b>	<b><u>17 COMPACTION</u></b>	
17.1.	<u>INTRODUCTION</u>	407
17.2.	<u>STANDARD PROCTOR TEST</u>	407
17.3.	MODIFIED PROCTOR TEST	410
17.4.	HARVARD MINIATURE COMPACTION TEST	410
17.5.	THE DIETERT TEST	410
17.6.	<u>ABBOT COMPACTION TEST</u>	411
17.7.	<u>JODHPUR MINI-COMPACTOR TEST</u>	411
17.8.	<u>FIELD COMPACTION METHODS</u>	412
17.9.	<u>PLACEMENT WATER CONTENT</u>	413
17.10.	<u>FIELD COMPACTION CONTROL</u>	413
17.11.	<u>FACTORS AFFECTING COMPACTION</u>	414
17.12.	<u>EFFECT OF COMPACTION ON SOIL PROPERTIES</u>	417
17.13.	<u>EXAMPLES FROM COMPETITIVE EXAMINATIONS</u>	420
17.14.	<u>LABORATORY EXPERIMENTS</u>	421

## **PART V**

### **STRENGTH AND STABILITY**

<b>CHAPTER</b>	<b><u>18 SHEAR STRENGTH</u></b>	
18.1.	INTRODUCTION	427
18.2.	THEORETICAL CONSIDERATIONS : MOHR'S STRESS CIRCLE	427
18.3.	MOHR-COULOMB FAILURE THEORY	430
18.4.	THE EFFECTIVE STRESS PRINCIPLE	432
18.5.	MEASUREMENT OF SHEAR STRENGTH	433
18.6.	DIRECT SHEAR TEST	434
18.7.	TRIAXIAL COMPRESSION TEST	436



18.8.	<u>UNCONFINED COMPRESSION TEST</u>	442
18.9.	<u>VANE SHEAR TEST</u>	450
18.10.	<u>SKEMPTON'S PORE PRESSURE PARAMETERS*</u>	452
18.11.	<u>SHEAR STRENGTH OF COHESIVE SOILS</u>	455
18.12.	<u>HVORSLEV SHEAR STRENGTH PARAMETERS</u>	461
18.13.	<u>STRESS PATH METHOD</u>	465
18.14.	<u>EXAMPLES FROM COMPETITIVE EXAMINATIONS</u>	470
18.15.	<u>LABORATORY EXPERIMENTS</u>	481
<b>CHAPTER 19</b>	<b><u>FAILURE ENVELOPES*</u></b>	
19.1.	<u>INTRODUCTION</u>	489
19.2.	<u>THE MOHR-COULOMB FAILURE ENVELOPE</u>	490
19.3.	<u>THE TRESCA FAILURE ENVELOPE</u>	492
19.4.	<u>VON MISES FAILURE ENVELOPE</u>	494
19.5.	<u>COHESIONLESS SOIL : KIRKPATRICK FAILURE SURFACE</u>	494
19.6.	<u>COHESIVE SOILS : FAILURE SURFACE STUDY BY WU, LOH AND MALVERN</u>	496
19.7.	<u>CONCLUSIONS</u>	498
<b>CHAPTER 20</b>	<b><u>EARTH PRESSURE</u></b>	
20.1.	<u>INTRODUCTION</u>	499
20.2.	<u>PLASTIC EQUILIBRIUM IN SOILS : ACTIVE AND PASSIVE STATES</u>	499
20.3.	<u>EARTH PRESSURE AT REST*</u>	503
20.4.	<u>ACTIVE EARTH PRESSURE : RANKINE'S THEORY</u>	504
20.5.	<u>ACTIVE EARTH PRESSURE OF COHESIVE SOILS</u>	510
20.6.	<u>PASSIVE EARTH PRESSURE : RANKINE'S THEORY</u>	512
20.7.	<u>RANKINE'S ACTIVE THRUST BY TRIAL WEDGES*</u>	521
20.8.	<u>COULOMB'S WEDGE THEORY</u>	522
20.9.	<u>REBHANN'S GRAPHICAL METHOD FOR ACTIVE PRESSURE</u>	524
20.10.	<u>CULMANN'S GRAPHICAL METHOD FOR ACTIVE PRESSURE</u>	532
20.11.	<u>EARTH PRESSURE COMPUTATION FOR PRACTICAL CASES</u>	535
20.12.	<u>DESIGN OF GRAVITY RETAINING WALL</u>	538
20.13.	<u>EXAMPLES FROM COMPETITIVE EXAMINATIONS</u>	541
<b>CHAPTER 21</b>	<b><u>BULKHEADS AND COFFERDAMS</u></b>	
21.1.	<u>INTRODUCTION : SHEET-PILE WALLS</u>	565
21.2.	<u>CLASSIFICATION OF BULKHEADS</u>	565
21.3.	<u>CANTILEVER SHEET PILE WALL</u>	567
21.4.	<u>CANTILEVER SHEET PILE WALL IN COHESIVE SOIL</u>	570
21.5.	<u>DESIGN OF ANCHORED BULKHEAD : FREE EARTH SUPPORT METHOD</u>	574
21.6.	<u>DESIGN OF ANCHORED BULKHEAD : FIXED EARTH METHOD</u>	577
21.7.	<u>DESIGN OF ANCHORS FOR BULKHEAD</u>	581
21.8.	<u>COFFER DAMS</u>	583
21.9.	<u>SOIL PRESSURE ON BRACED COFFER DAMS OR STRUTTED EXCAVATION</u>	585
21.10.	<u>EXAMPLES BASED ON COMPETITIVE EXAMINATIONS</u>	585
<b>CHAPTER 22</b>	<b><u>SHAFTS, TUNNELS AND CONDUITS*</u></b>	
22.1.	<u>STRESS DISTRIBUTION IN THE VICINITY OF SHAFTS</u>	591
22.2.	<u>STRESS DISTRIBUTION AROUND TUNNELS</u>	593



22.3.	ARCHING IN SOILS	594
22.4.	CLASSES OF UNDERGROUND CONDUITS	595
22.5.	<u>LOAD ON A DITCH CONDUIT</u>	<u>596</u>
22.6.	LOAD ON POSITIVE PROJECTING CONDUITS	597
22.7.	LOAD ON NEGATIVE PROJECTING CONDUIT	600
22.8.	IMPERFECT DITCH CONDUIT	601
22.9.	<u>LOAD ON CONDUIT DUE TO LIVE LOADS</u>	<u>602</u>
<b>CHAPTER</b>	<b><u>23 STABILITY OF SLOPES</u></b>	
23.1.	<u>INTRODUCTION</u>	<u>603</u>
23.2.	<u>STABILITY ANALYSIS OF INFINITE SLOPES</u>	<u>603</u>
23.3.	<u>STABILITY ANALYSIS OF FINITE SLOPES</u>	<u>608</u>
23.4.	<u>PLANAR FAILURE SURFACE : CULMANN'S METHOD</u>	<u>609</u>
23.5.	THE SWEDISH SLIP CIRCLE METHOD	611
23.6.	STABILITY OF SLOPES OF EARTH DAM	615
23.7.	<u>FRICTION CIRCLE METHOD</u>	<u>619</u>
23.8.	TAYLOR'S STABILITY NUMBER AND STABILITY CURVES	621
23.9.	BISHOP'S METHOD OF STABILITY ANALYSIS	632
23.10.	<u>EXAMPLES FROM COMPETITIVE EXAMINATIONS</u>	<u>635</u>
<b><u>PART VI</u></b>		
<b><u>FOUNDATION ENGINEERING</u></b>		
<b>CHAPTER</b>	<b><u>24 BEARING CAPACITY</u></b>	
24.1.	DEFINITIONS	639
24.2.	MINIMUM DEPTH OF FOUNDATION : RANKINE'S ANALYSIS	640
24.3.	TYPES OF BEARING CAPACITY FAILURES	641
24.4.	TERZAGHI'S ANALYSIS	643
24.5.	SKEMPTON'S VALUES FOR $N_c$	649
24.6.	GENERAL BEARING CAPACITY EQUATION : BRINCH HANSEN'S ANALYSIS	650
24.7.	MEYERHOF'S ANALYSIS	653
24.8.	VESIC'S BEARING CAPACITY EQUATION	654
24.9.	COMPARISON OF BEARING CAPACITY FACTORS	655
24.10.	EFFECT OF WATER TABLE ON BEARING CAPACITY	656
24.11.	EFFECT OF ECCENTRICITY OF LOADING	658
24.12.	I.S. CODE METHOD FOR COMPUTING BEARING CAPACITY	658
24.13.	PLATE LOAD TEST (IS 1888 : 1982)	673
24.14.	PENETRATION TESTS	680
24.15.	SAFE BEARING PRESSURE BASED ON TOLERABLE SETTLEMENT, $N$ VALUES AND $q_c$ VALUES	684
24.16.	PERMISSIBLE TOTAL AND DIFFERENTIAL SETTLEMENTS	686
24.17.	BEARING CAPACITY FROM BUILDING CODES	687
24.18.	<u>EXAMPLES FROM COMPETITIVE EXAMINATIONS</u>	<u>694</u>
<b>CHAPTER</b>	<b><u>25 SHALLOW FOUNDATIONS</u></b>	
25.1.	TYPES OF FOUNDATIONS	707
25.2.	SPREAD FOOTING	707
25.3.	SAFE BEARING PRESSURE	708



25.4.	SETTLEMENT OF FOOTINGS	709
25.5.	COMBINED FOOTING AND STRAP FOOTING	711
25.6.	MAT OR RAFT FOOTING	714
25.7.	I.S. CODE OF PRACTICE FOR DESIGN OF RAFT FOUNDATIONS	716
25.8.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	722

## CHAPTER 26 PILE FOUNDATIONS

26.1.	TYPES OF PILES	727
26.2.	PILE DRIVING	729
26.3.	LOAD CARRYING CAPACITY OF PILES	729
26.4.	DYNAMIC FORMULAE	730
26.5.	STATIC FORMULAE	732
26.6.	PILE LOAD TESTS	734
26.7.	PENETRATION TESTS	737
26.8.	GROUP ACTION IN PILES	737
26.9.	NEGATIVE SKIN FRICTION	740
26.10.	LATERALLY LOADED PILES*	744
26.11.	UNDER-REAMED PILE FOUNDATIONS	750
26.12.	BORED COMPACTION PILES	757
26.13.	CASED CAST- IN-SITU CONCRETE PILES	758
26.14.	UNCASED CAST-IN-SITU CONCRETE PILES	761
26.15.	BORED PILES	766
26.16.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	767

## CHAPTER 27 WELL FOUNDATIONS

27.1.	INTRODUCTION : CAISSONS	777
27.2.	SHAPES OF WELLS AND COMPONENT PARTS	777
27.3.	DEPTH OF WELL FOUNDATION AND BEARING CAPACITY	779
27.4.	FORCES ACTING ON A WELL FOUNDATION	780
27.5.	ANALYSIS OF WELL FOUNDATION	781
27.6.	HEAVY WELLS	785
27.7.	WELL CURB, CUTTING EDGE, STEINING AND BOTTOM PLUG	788
27.8.	WELL SINKING	791
27.9.	PNEUMATIC CAISSONS	794

## CHAPTER 28 MACHINE FOUNDATIONS

28.1.	SOIL DYNAMICS	797
28.2.	THE MASS SPRING SYSTEM	797
28.3.	VIBRATING SPRING-MASS SYSTEM WITH DAMPING	799
28.4.	FORCED VIBRATIONS	801
28.5.	TRANSMITTED FORCE	804
28.6.	NATURAL FREQUENCY OF FOUNDATION SOIL SYSTEM	804
28.7.	BARKEN'S METHOD	805
28.8.	BULB OF PRESSURE CONCEPT	806
28.9.	PAUW'S ANALOGY OF FOUNDATION SOIL SYSTEM : VERTICAL VIBRATIONS	807
28.10.	DYNAMIC ANALYSIS OF BLOCK FOUNDATIONS	813
28.11.	INDIAN STANDARD CODE OF PRACTICE FOR DESIGN OF FOUNDATIONS FOR RECIPROCATING TYPE MACHINES	817



28.12.	<u>INDIAN STANDARD CODE OF PRACTICE FOR DESIGN OF FOUNDATION FOR IMPACT TYPE MACHINES</u>	819
28.13.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	821

## PART VII PAVEMENT DESIGN

### CHAPTER 29 DESIGN OF FLEXIBLE PAVEMENT

29.1.	<u>INTRODUCTION : TYPES OF PAVEMENTS</u>	825
29.2.	<u>STRUCTURAL ELEMENTS OF A FLEXIBLE PAVEMENT</u>	826
29.3.	<u>STRESSES IN FLEXIBLE PAVEMENT</u>	826
29.4.	EQUIVALENT WHEEL AND AXLE LOADS	829
29.5.	DESIGN METHODS	830
29.6.	GROUP INDEX METHOD	831
29.7.	CALIFORNIA BEARING RATIO (CBR) METHOD	832
29.8.	NORTH DAKOTA METHOD	834
29.9.	BURMISTER'S DESIGN METHOD	835
29.10.	U.S. NAVY PLATE BEARING TEST METHOD	835
29.11.	LABORATORY EXPERIMENTS	837

### CHAPTER 30 DESIGN OF RIGID PAVEMENT

30.1.	INTRODUCTION	841
30.2.	STRESSES IN RIGID PAVEMENT	841
30.3.	STRESSES DUE TO WHEEL LOAD	842
30.4.	<u>STRESSES DUE TO WARPING</u>	843
30.5.	<u>STRESSES DUE TO SUB-GRADE FRICTION</u>	844
30.6.	<u>DESIGN METHOD</u>	846

### CHAPTER 31 STABILISATION OF SOILS

31.1.	<u>INTRODUCTION</u>	849
31.2.	<u>MECHANICAL STABILISATION</u>	849
31.3.	<u>CEMENT STABILISATION</u>	850
31.4.	<u>LIME STABILISATION</u>	853
31.5.	<u>BITUMEN STABILISATION</u>	854
31.6.	CHEMICAL STABILISATION	855
31.7.	STABILISATION BY HEATING	856
31.8.	ELECTRICAL STABILISATION	856

## PART VIII MISCELLANEOUS TOPICS

### CHAPTER 32 SITE INVESTIGATION AND SUB-SOIL EXPLORATION

32.1.	INTRODUCTION	859
32.2.	SITE RECONNAISSANCE	859
32.3.	SITE EXPLORATION	859
32.4.	METHODS OF SITE EXPLORATION	862
32.5.	SOIL SAMPLES AND SAMPLERS	864
32.6.	DISTURBED SAMPLING	866
32.7.	UNDISTURBED SAMPLING	866



32.8.	PENETRATION AND SOUNDING TESTS	867
32.9.	GEOPHYSICAL METHODS	869
<b>CHAPTER 33    ADVANCED MEASURING INSTRUMENTS</b>		
33.1.	ELECTRONIC CONSOLIDATION APPARATUS	873
33.2.	ELECTRONIC DIRECT SHEAR APPARATUS	873
33.3.	ELECTRONIC TRIAXIAL SHEAR TEST EQUIPMENT	874
33.4.	HYDRAULIC EARTH PRESSURE CELL	874
33.5.	VIBRATING-WIRE EARTH PRESSURE CELLS	876
33.6.	VIBRATING-WIRE EXTENSOMETER	878
33.7.	VIBRATING-WIRE PORE PRESSURE MEASUREMENT SYSTEM	879
33.8.	THE GDS CONSOLIDATION LOGGING SYTEM	880
33.9.	HYDRAULICALLY OPERATED CONSOLIDATION CELLS	882
33.10.	THE GDS 1000 cc/2000 kPa DIGITAL CONTROLLER	884
33.11.	HYDRAULICALLY OPERATED TRIAXIAL CELL	884
<b>CHAPTER 34    REINFORCED EARTH AND GEOTEXTILES</b>		
34.1.	REINFORCED EARTH	887
34.2.	DESIGN OF REINFORCED EARTH WALL	889
34.3.	GEOTEXTILES : DEFINITION AND TYPES	892
34.4.	FUNCTIONS OF GEOTEXTILES	894
34.5.	USE OF GEOTEXTILES IN EARTH DAM CONSTRUCTION	898
34.6.	USE OF GEOTEXTILES IN ROAD WORKS	899
34.7.	USE OF GEOTEXTILES IN RAILWAYS WORKS	900
34.8.	USE OF GEOTEXTILES IN EROSION CONTROL	901
34.9.	USE OF GEOTEXTILES IN BEARING CAPACITY IMPROVEMENT	902
34.10.	STORAGE, HANDLING AND PLACEMENT OF GEOTEXTILES	903
<b>REFERENCES</b>		905
<b>INDEX</b>		911



## LIST OF EXPERIMENTS

<i>S. No.</i>	<i>Experiment</i>	<i>Page</i>
1.	Determination of water content by oven drying method	... 87
2.	Determination of specific gravity by density bottle	... 89
3.	Determination of specific gravity by pycnometer	... 90
4.	Determination of water content by pycnometer	... 90
5.	Determination of dry density by water displacement method	... 91
6.	Determination of field density by core cutter method	... 92
7.	Determination of field density by sand replacement method	... 93
8.	Determination of grain size distribution by sieving	... 95
9.	Determination of grain size distribution by hydrometer	... 98
10.	Determination of grain size distribution by pipette	... 101
11.	Determination of liquid limit of soil	... 104
12.	Determination of plastic limit of soil	... 106
13.	Determination of shrinkage factors of soil	... 107
14.	Determination of permeability by constant head test	... 201
15.	Determination of permeability by falling head test	... 203
16.	Determination of consolidation properties	... 389
17.	Determination of compaction properties	... 421
18.	Determination of shear parameters by direct shear test	... 481
19.	Determination of unconfined compressive strength of soil	... 484
20.	Determination of shear parameters by triaxial test	... 485
21.	Determination of California bearing ratio	... 837



# Symbols

$A$ = Pore pressure parameter ; area, constant	$c'$ = Effective unit cohesion	$f_c$ = Unit skin friction of shaft of penetrometer
$\bar{A}$ = Pore pressure parameter	$c_c$ = Critical damping factor	$f_n$ = Natural frequency
$\text{\AA}$ = Angstrom	$c_d'$ = Cohesion in drained test	$f_s$ = Unit skin friction
$A_f$ = Parameter $A$ at failure	$c_e$ = Hvorslev true cohesion	$G$ = Specific gravity of soil particles ; centre of gravity
$A_p$ = Sectional area of pile toe	$c_m$ = Mobilised cohesion	$G_k$ = Specific gravity of kerosene
$A_v$ = Area of voids	$c_u$ = Apparent cohesion	$G_m$ = Mass specific gravity of soil mass
$A_s$ = Amplitude of vibrations	$c_v$ = Coefficient of consolidation	$g$ = Acceleration due to gravity
$A_{sp}$ = Permissible amplitude	$c$ = Diameter ; depth ; distance	$H$ = Height ; thickness ; depth total hydraulic head
$a$ = Area ; constant ; radius	$D_a$ = Diameter of average particle	$H_a$ = Height of air in specimen
$a_c$ = Air content	$D_{10}$ = Effective size ; 10 percent finer size	$H_c$ = Critical height (depth) of slope or cut
$B$ = Pore pressure parameter ; constant ; width	$D_F$ = Depth factor	$H_s$ = Effective height in hydro- meter analysis, sampling depth
$\bar{B}$ = Pore pressure parameter ;	$D_P$ = Depth of test plate	$H_s$ = Height of soil solids
$B_p$ = Width of test plate	$D_{min}$ = Minimum depth of foun- dation	$H_w$ = Height of water
$b$ = Constant ; width ; thick- ness of aquifer	$d$ = Diameter ; distance	$h$ = Hydraulic head ; height ; thickness
$C$ = composite correction (hyd- rometer) ; constant ; total cohesion	$E$ = Young's modulus of elas- ticity ; electric potential	$h_c$ = Capillary rise ; Henry's constant of solubility of air in water
$C_e$ = Compression index ; coeffi- cient of curvature	$e$ = Voids ratio ; coefficient of restitution ; eccentricity ; base of natural logarithms (2.7183)	$h_w$ = Piezometric head
$C_d$ = Dispersing agent correction	$e_w$ = Water voids ratio	$I$ = Current ; Index
$C_m$ = Meniscus correction	$e_{max}$ = Maximum voids ratio	$I_D$ = Density index
$C_s$ = Expansion index ; shape constant	$e_{min}$ = Minimum voids ratio	$i$ = Flow index ; Impact factor
$C_t$ = Temperature correction	$F$ = Force ; factor of safety ; de- flection factor ; function of	$I_L$ = Liquidity index
$C_u$ = Uniformity coefficient ; coefficient of elastic uni- form compression of soil	$F_c$ = Factor of safety with res- pect to cohesion	$I_T$ = Toughness index
$C_v$ = Coefficient of volume compressibility	$F_0$ = Dynamic force	$i$ = Hydraulic gradient
$c$ = Unit cohesion, damping factor	$F_T$ = Transmitted Force	$i_c$ = Critical hydraulic gradient
	$f$ = Silt factor ; stress ; function of ; operating frequency	$i_e$ = Exit gradient



$J$ = Seepage force	$m$ = Mass; modulus ; ratio ; coefficient	$Q_{up}$ = Failure load ; ultimate bearing capacity of pile
$j$ = Seepage per unit volume	$m'$ = Complimentary modulus	$Q_{ug}$ = Ultimate bearing capacity of pile group
$K$ = Coefficient ; coefficient of earth pressure ; complete elliptic integral of the first kind with modulus $m$ ; Influence factor	$m_s$ = Equivalent soil mass	$q$ = Unit surcharge ; intensity of uniformly distributed load ; rate of flow or unit discharge
$K'$ = Complete elliptic integral of the first kind with modulus $m'$	$m_c$ = Coefficient of volume change ; mass of vibrator	$q_c$ = Static cone resistance, North Dakota cone bearing value
$K_a$ = Coefficient of active earth pressure	$N$ = Normal force ; percent finer ; number of blows ; integer	$q_n$ = Net foundation pressure
$K_B$ = Boussinesq influence factor for vertical pressure	$N_c, N_q, N_\gamma$ = Bearing capacity factors (general shear failure)	$q_f$ = Ultimate bearing capacity
$K_0$ = Coefficient of earth pressure at test	$N'_c, N'_q, N'_\gamma$ = Bearing capacity factors (local shear failure)	$q_u$ = Unconfined compressive strength
$K_p$ = Coefficient of passive earth pressure	$N_d$ = Number of potential drops	$R$ = Corrected hydrometer reading ; radius; radius of zero drawdown ; reaction ; resistance ; dial reading ; amplitude ratio ; maximum scour depth
$K_w$ = Westergaard's influence factor for vertical pressure	$N_f$ = Number of flow channels	$R_H$ = Hydraulic radius
$k$ = Coefficient of permeability; spring constant; modulus	$N_{\gamma q}$ = Resultant bearing capacity factor (Meyerhof)	$R_h'$ = Hydrometer reading at top of meniscus
$k'$ = Effective permeability in a transformed flow net	$n$ = Porosity ; number ; exponent ; ratio	$R_h$ = Hydrometer reading at bottom of meniscus, graduation reading on hydrometer stem
$k_p$ = Coefficient of percolation	$n_a$ = Percentage air voids	$R_f$ = Total ultimate skin friction
$k_s$ = Modulus of subgrade reaction	$P$ = Force; load; weight	$R_L$ = Lacey's normal scour depth
$k_{physical}$ = Physical permeability coefficient	$P_a$ = Resultant active pressure	$R_p$ = Total ultimate point bearing resistance
$L$ = Length; distance	$P_0$ = Equivalent wheel load	$R_T$ = Total tension (pull)
$\bar{L}$ = Length of chord	$P_p$ = Resultant passive load	$r$ = Radius; distance, frequency ratio
$L$ = Length of arc	$p$ = Pressure ; reaction ; percentage	$r_f$ = Unit skin friction
$l$ = Length; distance	$p_a$ = Intensity of active earth pressure ; atmospheric pressure	$r_p$ = Unit point resistance
$M$ = Moment; factor	$p_p$ = Intensity of passive earth pressure	$r_u$ = Pore-pressure ratio
$M_D$ = Driving moment	$pF$ = Logarithm of soil suction	
$M_0$ = Overturning moment	$p_s$ = Seepage pressure	
$M_R$ = Resisting moment	$Q$ = Total quantity of flow or discharge ; load	
	$Q_d$ = Allowable load	



$S$ = Degree of saturation; storage coefficient ; shear force ; real set per blow	$v_s$ = Seepage velocity	$\gamma_{sat}$ = Saturated density
$S_n$ = Stability number	$W$ = Weight; total weight of soil	$\gamma'$ = Submerged density
$S_R$ = Specific retention	$W_a$ = Weight of anvil	$\gamma_s$ = Density of soil solids
$S_s$ = Specific surface	$W_b$ = Weight of foundation block	$\gamma_w$ = Density of water
$S_Y$ = Specific yield	$W_d$ = Dry weight	$\Delta$ = Change or increment (or decrement); triangle
$s$ = Soil suction; drawdown	$W_D$ = Weight of soil solids per millilitre of suspension smaller than diameter $D$	$\delta$ = Angle of wall friction ; elastic deflection of machine foundation
$T$ = Torque ; tangential com- ponent ; temperature co- efficient of transmissibility	$W_f$ = Weight of frame	$s$ = Static deflection of spring
$T_s$ = Surface tension	$w$ = Water content; complex potential	$\epsilon$ = strain
$T_v$ = Time factor	$w_f$ = Water content at failure	$e$ = Base of natural logarithm (=2.7183)
$t$ = time; thickness	$w_L$ = Liquid limit	$\eta$ = Viscosity ; coordinate variable
$U$ = Degree of consolidation; total pore pressure ; Jacobi's symbol for the elliptic in- tegral of the first kind	$w_P$ = Plastic limit	$\eta_b$ = Efficiency of blow
$u$ = Pore pressure ; stream function	$w_S$ = Shrinkage limit	$\eta_a$ = Efficiency of pile group
$\bar{u}$ = Excess hydrostatic pressure	$w_{sat}$ = Water content at full saturation	$\eta_h$ = Efficiency of hammer
$V$ = Volume ; total volume of soil ; velocity of wave	$x$ = Abscissa ; coordinate ; distance	$\sigma_1', \sigma_2', \sigma_3'$ = Effective princi- pal stress
$V_a$ = Volume of air	$\bar{x}$ = Distance	$\theta$ = Angle; inclination
$V_d$ = Volume of dry soil	$y$ = Ordinate ; coordinate ; distance	$\theta_f$ = Inclination of failure plane
$V_h$ = Volume of hydrometer	$\bar{y}$ = Distance	$\mu$ = Poisson's ratio ; micron ; magnification factor
$V_p$ = Volume of pipette	$Z$ = Depth ; thickness ; dis- tance ; complex coordinate	$\pi = 3.1416$
$V_s$ = Volume of solids ; seepage velocity	$\bar{Z}$ = Distance	$\Pi(U, a)$ = Jacobi's elliptic in- tegral of third kind
$V_p$ = Volume of voids	$Z_c$ = Depth of tension crack	$\Pi(n, \phi, k)$ = Legendre's ell- iptic integral of third kind
$V_w$ = Volume of water	$z$ = Coordinate ; variable ; depth ; thickness; height ; position head; complex coordinate	$\rho$ = Settlement, deflection ; penetration ; resistivity ; horizontal displacement of well foundation
$V_{wY}$ = Volume of water drained by gravity	$\alpha$ = Angle	$\rho_c$ = Corrected, final consolid- ation settlement
$V_{wR}$ = Volume of water retained	$\alpha_A, \alpha_B$ = Directional angles	$\rho_f$ = Final settlement
$v$ = Velocity of flow ; superfical velocity ; pressure function	$\beta$ = Angle; angle of obliquity	$\rho_p$ = Settlement of test plate
$v_c$ = Critical velocity	$\gamma$ = bulk density; variable	$\Sigma$ = Summation of
	$\gamma_d$ = Dry density	



$\sigma$ = Total pressure; normal stress	$\sigma_x, \sigma_y, \sigma_z$ = Stress component in $x, y, z$ directions	$\phi_e'$ = Hvorslev true friction angle
$\sigma'$ = Effective pressure (stress)	$\tau$ = Shear stress ; tangential stress	$\phi_\mu$ = Mobilised $\phi$
$\sigma_1, \sigma_2, \sigma_3$ = Major, intermediate and minor principal stresses	$\tau_f$ = Shear strength	$\phi_w$ = Weighted friction angle
$\phi_u$ = Apparent angle of shearing resistance	$\Phi$ = Hydraulic potential ; Stress function	$\phi_x$ = % pressure at any point
$\sigma_d$ = Deviator stress	$\phi$ = Angle; angle of shearing resistance, velocity potential ; am U	$\psi$ = Angle; Stream function
$\sigma_c'$ = Consolidation pressure	$\psi'$ = Effective angle of shearing resistance	$\omega$ = Angle ; operating frequency
$\sigma_p'$ = Preconsolidation pressure	$\phi_d'$ = Angle of shearing resistance in drained test	$\omega_n$ = Natural frequency
$\sigma_v$ = Vertical pressure, normal		$\infty$ = Infinity
		$\zeta$ = Complex coordinate
		$\xi$ = Coordinate variable ; damping ratio

**PART I**

**ELEMENTARY PROPERTIES**

- 1. INTRODUCTION**
- 2. PRELIMINARY DEFINITIONS AND RELATIONSHIPS**
- 3. DETERMINATION OF INDEX PROPERTIES**
- 4. CLASSIFICATION OF SOILS**
- 5. SOIL STRUCTURE AND CLAY MINERALOGY**



# Introduction

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## 1.1. SOIL AND SOIL ENGINEERING

The term '*Soil*' has various meanings, depending upon the general professional field in which it is being considered. To an agriculturist, soil is the substance existing on the earth's surface, which grows and develops plant life. To the geologist also, soil is the material in the relatively thin surface zone within which roots occur, and all the rest of the crust is grouped under the term *rock* irrespective of its hardness. To an engineer, soil is the unaggregated or uncemented deposits of mineral and/or organic particles or fragments covering large portion of the earth's crust. It includes widely different materials like boulders, sands, gravels, clays and silts, and the range in the particle sizes in a soil may extend from grains only a fraction of a micron ( $10^{-4}$  cm) in diameter upto large size boulders.

Soil engineering, *Soil Mechanics* or Geotechnique is one of the youngest disciplines of civil engineering involving the study of soil, its behaviour and application as an engineering material. According to Terzaghi (1948) : "Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rocks regardless of whether or not they contain an admixture of organic constituent". The term *Soil Engineering* is currently used to cover a much wider scope implying that it is a practical science rather than a purely fundamental or mathematical one. The term *Foundation Engineering* is a branch of civil engineering, which is associated with the design, construction, maintenance, and renovation of footings, foundation walls, pile foundations, caissons, and all other structural members which form the foundations of buildings and other engineering structures (Taylor, 1948).

Soil is considered by the engineer as a complex material produced by the weathering of the solid rock. The formation of soil is as a result of the geologic cycle continually taking place on the face of the earth. The cycle consists of weathering or denudation, transportation, deposition and upheaval, again followed by weathering, and so on. *Weathering* is caused by the physical agencies such as periodical temperature changes, impact and splitting action of flowing water, ice and wind, and splitting actions of ice, plants and animals. Cohesionless soils are formed due to physical disintegration of rocks. Chemical weathering may be caused due to oxidation, hydration, carbonation and leaching by organic acids and

water. Clay minerals are produced by chemical weathering. Soil obtained due to weathering may be residual or transported. *Residual* soils, which remain in place directly over the parent rock, are relatively shallow in depth. The deposits of the *transported soils* may be considerable in depth and their homogeneity or heterogeneity depends upon the manner of their transportation and deposition. The various agencies of transporting and redepositing soils are : water, ice, wind and gravity. Water-formed transported soils are termed as *alluvial, marine* or *lacustrine*. All the material, picked up, mixed, disintegrated, transported and redeposited by glaciers either by ice or by water issuing from melting of glaciers, is termed *glacial drift* or *simple drift*. The glacial deposits in general consists of a heterogeneous mixture of rock fragments and soils of varying sizes and proportions and, except the stratified drift deposited by glacial streams, are without any normal stratification. Dune sand and loess are the wind-blown (*aeoline*) deposits. *Loess* is the wind-blown silt or silty clay having little or no stratification. Soils transported by gravitational forces are termed *colluvial soils*, such as *talus*. The accumulation of decaying and chemically deposited vegetable matter under conditions of excessive moisture results in the formation of *cumulose* soils, such as *peat* and *muck*.

## 1.2. HISTORY OF DEVELOPMENT OF SOIL MECHANICS

The knowledge of the use of soil extends into prehistoric times, when man started constructing dwellings for living and roads for transportation. In the more primitive civilisations, soil was used by man as a construction material for foundations of structure and for the structures themselves. The knowledge of soils for the foundations, bunds and roads was gained by trial and error experiences. Through ancient times and even within the last few generations practically all improvement was the result of a continuously broadening by empirical knowledge.

The use of both timber and stone caissons of soft-ground shaft construction was known in Egypt in 2000 B.C. The cutting edge was made of a round limestone block with a vertical hole bored into its middle. The outside surface of the caisson was made smooth for reasons of reducing sinking resistance caused by friction. One of the greatest structures in ancient times was the famous 'hanging garden' built by the Babylonian King Nebuchadnezzar. The big retaining walls to support the terraces of the garden required some knowledge of earth pressures, even if the knowledge was empirical (Jumikis, 1962). The technical literature of the time during the Roman Empire supplies ample evidence that the Romans paid much attention to some properties of soils, and to the stability of foundations. The Romans built notable engineering structures, such as : harbours, moles, break-waters, aqueducts, bridges, large public buildings, sewage lines and a vast network of durable and excellent roads, requiring solutions of earth work and foundation design. The Roman engineer Vitruvius wrote his *Ten books on Architecture* sometimes in the first century B.C. Discussing the stability of buildings, Vitruvius writes that "..... greatest care must be taken in the substructure, because, in these, immense damage is caused by the earth piled against them. For it cannot remain of the same weight as it usually has in the summer ; it swells in the winter by absorbing water from the rains. Consequently, by its weight and expansion it bursts and thrusts out the retaining wall"-(Jumikis, 1962). For foundations in loose or



marshy land, he recommends the use of 'piles to be driven close together by machinery, and the intervals between them to be filled with charcoal.' In India also, Mansar, Mayamata, Visvakarma, Agastya, Santakumara, Mandana, Srikumara, etc. wrote books laying down rules of construction. Among these, Mansar's 'Silpa Sastra,' written sometimes in sixth or seventh century, became very popular. Mansar recommended compaction of soil by cows and oxen, and dewatering of foundations.

Many structures were built during the medieval period (about 400 to 1400 A.D.). One of the main problems they had was about the compression of soil and the consequent settlement of buildings. During the past centuries, the compressible soil upon which heavy structures, such as cathedrals etc. were built had enough time to consolidate, causing large settlements. The Leaning Tower of Pisa, constructed between 1174 to 1350 A.D., is one such example. In India, Taj Mahal was constructed between 1632 to 1650. It had unique foundation problems because of its proximity of the river Jamuna. The terrace and the mausoleum building, as well as the minarets, rest on one firm, compact bed of masonry, supported on masonry cylindrical wells sunk at close intervals. In the field of earth dams, the most notable example reported is that of Mudduk Masur dam in South India, of 33 m height, and built in 1500 A.D. (Oza, 1969).

In 1661, France undertook an extensive public works programme in improving the highways, and the building of canals. In the later part of the 17th century, French Military engineers contributed some empirical and analytical data pertaining to earth pressure on retaining walls for the design of revetments of fortifications. France established a Department of Roads and Bridges in 1715, and in 1747, the Famous *Ecole des ponts et chaussées* was started. The first major contribution to the present scientific study of soil behaviour may be traced back towards the end of the eighteenth century, when Coulomb (1776), a Frenchman, published his wedge theory of earth pressure. Coulomb was the first to introduce the concept that shearing resistance of soil is composed of two components, namely, cohesion and friction. Poncelet (1788-1867), a famous geometer, extended Coulomb's theory, giving a graphical method of finding the magnitude of earth pressure on the wall, vertical as well as for inclined wall surface on the backfill side, and for arbitrary broken polygonal surfaces. K.Culmann (1866) gave the Coulomb-Poncelet theory a geometrical formulation. The earth pressure theory was elaborated by graphical analysis also by Rebhann (1871) and Weyrauch (1878). Two important laws—Darcy's law for flow of water through soils and Stoke's law for settlement of soil particles in liquid—were put forward in 1856. Even today, these laws play an important role in soil engineering. In 1857, Rankine presented his theory for calculating earth pressure and safe bearing capacity of foundation. Rankine and other workers of his time did not take cohesion of clay soil into calculations, although they knew its existence. Another important contribution in the nineteenth century was made by Boussinesq (1885) who gave his analysis for stress distribution in a semi-infinite, elastic medium under surface point loads. To test the earth pressure theories, Muller-Breslau (1906) performed some relatively extensive and elaborate experiments with a large scale model retaining wall.

In 1871, O. Mohr gave a graphical representation of stress at a point, popularly

known as Mohr's stress circle. In soil mechanics, Mohr's stress circles are extensively used in the analysis of the shearing strength of soils. It is only in the beginning of the twentieth century that the basic physical properties of soil in general were understood, and the work of Atterberg, a Swedish soil scientist, and that of the Geotechnical Commission of the Swedish Government under the chairmanship of Dr. Fellenius, in this direction are remarkable. Atterberg was the first to propose in 1911 the different stages of consistency in which a clay soil may exist, depending upon its water content. To measure the shear strength of sand, shear box was probably first developed in France, by Leygue in about 1885. Later, it was improved by Krey (1918) in Germany, and Terzaghi and Casagrande in U.S.A. Resal (1910) and Bell (1915) are credited to have extended Rankine's analysis of earth pressure so as to include soil with both friction and cohesion. Bell also suggested a method of calculating the bearing capacity of cohesive soils. In 1916, Petterson and Hultin used the circular sliding theory with the so-called friction circle in stability calculations. This method was further developed by Fellenius in 1926, and is now known as the Swedish method of slope analysis. In 1913, the Swedish Geotechnical Commission was appointed, with Fellenius as its chairman.

In 1920, L. Prandtl gave his theory of plastic equilibrium, which forms the basis of various bearing capacity theories developed later. Dr. Terzaghi published his theory of consolidation in 1923 and the term *Soil Mechanics* was coined by him in 1925 when his book under the equivalent German title *Erdbaumechanik* was published. Dr. Terzaghi's contributions in the field of soil engineering have been immense and he is fittingly called the 'Father of Soil Mechanics'. Another important contribution made recently (1933) is that of Proctor on the principles of soil compaction. In 1922-23 Pavlovsky in Russia solved the complex problems of seepage below the hydraulic structures, and gave the electrical analogy method for the seepage computations. However, since his work was in Russian language, it remained unknown to the English literature till 1933, Weaver (1934) and Khosla (1936) solved some of the seepage problems independently. During World War II (1939-45) and after, a great impetus to the development of soil engineering has been made by various scientists and engineers of different countries of the World, and today it is recognised as a well-established branch of engineering. Several International conferences on Soil Mechanics and Foundation Engineering have been held till now under the auspices of International Society of Soil Mechanics and Foundation Engineering such as at Harvard (Massachusetts, U.S.A.) 1936, Rotterdam (Netherlands) 1948, Zurich (Switzerland) 1953, London (U.K.) 1957, Paris (France) 1961, Montreal (Canada) 1965, Mexico city (Mexico) 1969, Moscow (U.S.S.R.) 1973, Tokyo (Japan) 1977, Stockholm (Sweden) 1981, San Fransisco (U.S.A) 1985, Riode Jeneiro (Brazil) 1989, New Delhi (India) 1994, etc. etc.

### 1.3. FIELD OF SOIL MECHANICS

The field of soil mechanics is very vast. The civil engineer has many diverse and important encounters with soil. Apart from the testing and classification of various types of soils in order to determine its physical properties, the knowledge of soil mechanics is particularly helpful in the following problems in civil engineering.

1. **Foundation design and construction.** Foundation is an important element of all



civil engineering structures. Every structure — building, bridge, highway, tunnel, canal or dam — is founded in or on the surface of the earth. It is, therefore, necessary to know the bearing capacity of the soil, the pattern of stress distribution in the soil beneath the loaded area, the probable settlement of the foundation, effect of ground water and the effect of vibrations, etc. The suitability of various types of foundations — *i.e.*, spread foundation, pile foundation, well foundation etc. — depend upon the type of soil strata, the magnitude of loads and ground water conditions. A knowledge of shrinkage and swelling characteristics of soil beneath the foundation is also very essential.

**2. Pavement design.** A pavement can either be flexible or rigid, and its performance depends upon the subsoil on which it rests. The thickness of a pavement and its component parts, depends upon some certain characteristics of the subsoil, which should be determined before the design is made. On busy pavements, where the intensity of traffic is very high, the effect of repetition of loading and the consequent fatigue failure has to be taken into account. Apart from these, other problems of pavement design are : frost, heave and thaw with their associated problems of frost damage to pavements ; frost penetration depth ; remedial measures to prevent frost damage ; problems of 'pumping' of clay subsoils and suitability of a soil as a construction material for building highways or railways, earth fills or cuts etc. A knowledge of the techniques for the improvement of the soil properties such as strength and stability is very much helpful in constructing pavements on poor soils by stabilising them.

**3. Design of underground structures and earth retaining structures.** The design and construction of underground (subterranean) and earth retaining structures constitute an important phase of engineering. The examples of underground structures include tunnels, underground buildings, drainage structures and pipe lines. The example of earth retaining structures are : gravity retaining wall, anchored bulk heads and cofferdams. A knowledge of *soil structure interaction* is essential to design properly such structures subjected to soil loadings.

**4. Design of embankments and excavations.** When the surface of the soil structure is not horizontal, the component of gravity tends to move the soil downward, and may disturb the stability of the earth structure. A thorough knowledge of shear-strength and related properties of soil is essential to design the slope and height (or depth) of the embankment (or excavation). The possibility of the seeping ground water reducing the soil strength while excavating must also be taken into account. It may sometimes be essential to drain the subsoil water, to increase the soil strength and to reduce the seepage forces. Deep excavations require lateral braces and sheet walls to prevent caving in.

**5. Design of earth dams.** The construction of an earth dam requires a very thorough knowledge of whole of the Soil Mechanics. Since soil is used as the only construction material in an earth dam, which may either be homogeneous or of composite section, its design involves the determination of the following physical properties of soil : index properties such as density, plasticity characteristics and specific gravity, particle size distribution and gradation of the soil; permeability, consolidation and compaction characteristics, and shear strength parameters under various drainage conditions. Since huge earth mass is involved

in its construction, suitable soil survey to the nearby area may be essential for the borrow-pit area. The determination of the optimum water content at which maximum density will be obtained on compaction, is probably the most essential aspect of the design. Apart from the seepage, characteristics of the dam section must be thoroughly investigated since these have the greatest impact on the stability of the slopes as well as the foundations of the dam. The consolidation characteristics help in predicting the long range behaviour of the dam toward settlement and the consequent reduction in the pore pressure. Lastly, the possible effect of vibrations during an earthquake should also be taken into account while designing.

The performance of the soil in the designs cited above depends upon the characteristics of soil. Therefore, the testing of soil with relation to the determination of its physical properties, and the evaluation of effects of certain other factors such as seepage conditions etc., forms the most essential part of the development of soil engineering. It is through research only that design and construction methods are modified to give maximum safety and/or economy, and new methods are evolved. The knowledge of theoretical soil mechanics, assuming the soil to be an ideal elastic isotropic and homogeneous material, helps in predicting the behaviour of the soil in the field.

#### 1.4. SI UNITS

In this text book, SI units have been used throughout. However, prior to this, metric units were in use, not only in India, but in several parts of the World. Most of the equipments in these laboratories have the use of metric units. These equipments use grams and kilograms. Hence *mass* units of grams and kilograms (formerly called 'weight') will continue to be used. *Also, the term 'mass' and 'weight' are commonly used interchangeably.* This may cause some problem with 'unit weight' terms. In this text, '*mass*' will be represented in terms of grams and kilograms while '*weight*' will be represented in terms of Newtons and kilo Newtons. The term *density* is defined as mass per unit volume; hence it will be represented in terms of  $\text{g/cm}^3$  or  $\text{kg/m}^3$  units. On the contrary, the term '*unit weight*' is the weight per unit volume. Hence it will be represented in terms of  $\text{kN/m}^3$ .

The value of gravitational constant  $g$  will be taken as  $981 \text{ cm/s}^2$ , for computational purposes.

Thus, in order to convert the density (expressed in terms of  $\text{g/cm}^3$ ) into unit weight, multiply the former by 9.81. For example, if a soil mass has a mass of 216 g and volume of  $120 \text{ cm}^3$ ,

$$\text{Density} = \frac{\text{Mass}}{\text{Vol.}} = \frac{216}{120} = 1.8 \text{ g/cm}^3$$

$$\text{and unit weight} = \frac{\text{Weight}}{\text{Vol.}} = \frac{216}{120} \times 9.81 = 17.66 \text{ kN/m}^3$$

*Pressure*, which is defined to be a force per unit area, will be expressed in terms of  $\text{kN/m}^2$  or in term of Kilopascals, kPa.



## Preliminary Definintions and Relationships

### 2.1. SOIL AS A THREE PHASE SYSTEM

A soil mass is a *three phase system* consisting of solid particles (called soil grains), water and air. The void space between the soil grains is filled partly with water and partly with air. However, if we take a dry soil mass, the voids are filled with air only. In case of a perfectly saturated soil, the voids are filled completely with water. In general, the soil mass has three constituents which do not occupy separate spaces but are blended together forming a complex material [Fig. 2.1 (a)] the properties of which depend upon the relative percentages of these constituents, their arrangement and a variety of other factors. For calculation purposes, it is always more convenient to show these constituents occupying separate spaces, as shown in [Fig. 2.1(b)(i) and (b)(ii)].

As shown in Fig. 2.1 (b) (i), the total volume  $V$  of the soil mass consists of (i) volume of air  $V_a$ , (ii) volume of water  $V_w$  and (iii) the volume of solids  $V_s$ . The volume of voids  $V_v$ , is, therefore, equal to volume of air plus the volume of water. Similarly, Fig. 2.1 (b) (ii) shows the weights. The weight of air is considered to be negligible. Hence the weight of total voids is equal to the weight of water  $W_w$ . The weight of solids is represented by  $W_d$  (or

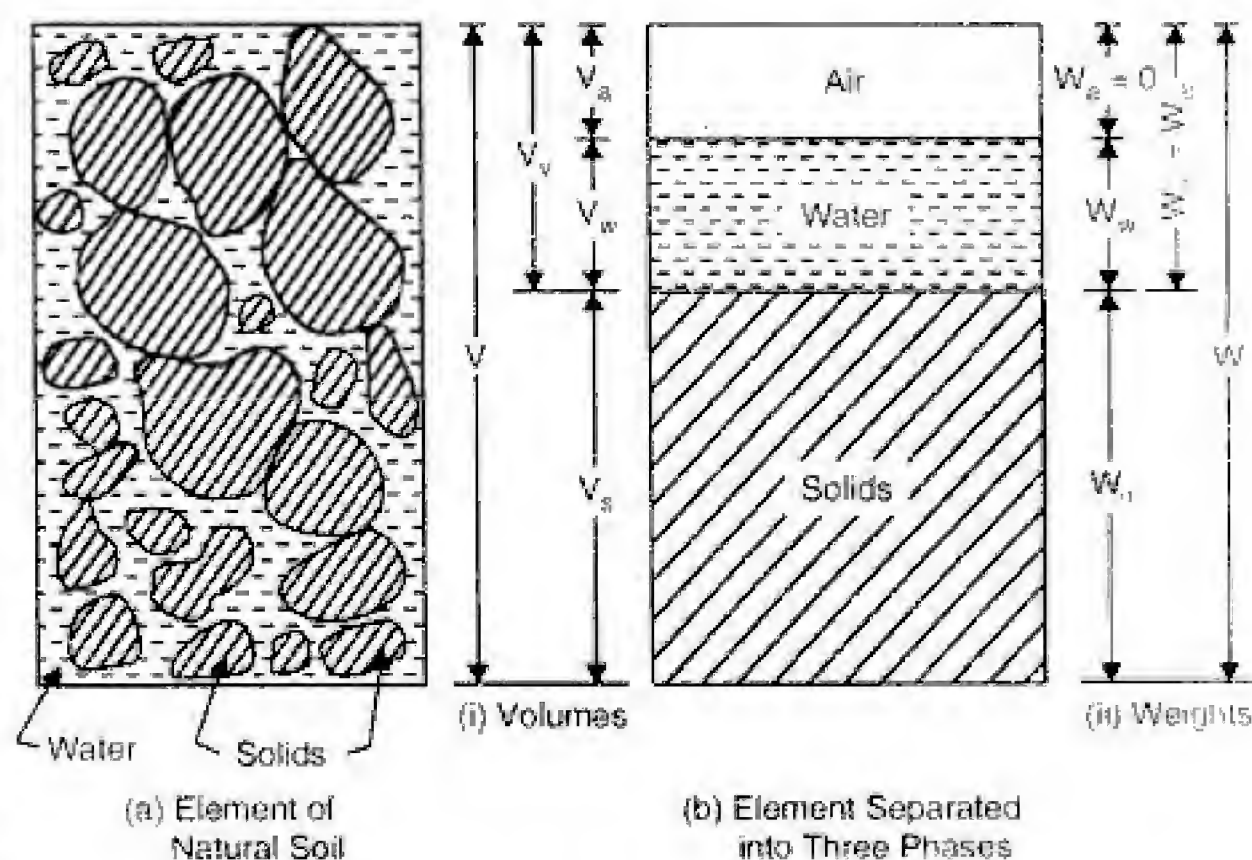


FIG. 2.1. SOIL AS A THREE PHASE SYSTEM

$W_s$ ) which is evidently equal to the dry weight of soil sample. The total weight  $W$  of the moist sample is, therefore, equal to  $(W_w + W_d)$ .

## 2.2. WATER CONTENT, DENSITY AND UNIT WEIGHTS

(a) **Water content.** The water content  $w$ , also called the *moisture content*, is defined as the ratio of weight of water  $W_w$  to the weight of solids ( $W_s$  or  $W_d$ ) in a given mass of soil :

$$w = \frac{W_w}{W_d} \times 100 \quad \dots(2.1)$$

The water content is generally expressed as a percentage. However, when used in the formulae giving relationship between certain quantities, it may also be expressed as a fraction. Rewriting Eq. 2.1, we have

$$w = \frac{W - W_d}{W_d} \times 100 = \left( \frac{W}{W_d} - 1 \right) 100 \quad \dots(2.1a)$$

The usual procedure to find the natural water content is to take a mass of about 20 to 30 g of soil sample in a container and determine its mass  $M$  very accurately. The soil sample is then kept in an oven ( $105^\circ - 110^\circ \text{C}$ ) for about 24 hours so that it becomes perfectly dry. Its dry mass  $M_d$  is then determined and the water content is calculated from the relation,

$$w = \frac{M_w}{M_d} \times 100 = \frac{M - M_d}{M_d} \times 100 = \left( \frac{M}{M_d} - 1 \right) 100 \quad \dots (2.1b)$$

In the field of geology and agriculture, water content ( $w'$ ) is defined as the ratio of the weight of water to the total weight of soil mass. Thus,

$$w' = \frac{W_w}{W} \times 100 = \frac{M_w}{M} \times 100 \quad \dots(2.1 c)$$

In order to find relation between  $w$  and  $w'$ , we have from eq. 2.1 (c)

$$w' = \frac{M_w}{M_w + M_d} \quad \text{or} \quad \frac{1}{w'} = \frac{M_w + M_d}{M_w} = 1 + \frac{1}{w}$$

From which we get  $w = \frac{w'}{1 - w'} \quad \dots(2.1 d)$

Expressed as percentage,  $w = \frac{w'}{100 - w'} \quad \dots(2.1 e)$

(b) **Density of soil.** The density of soil is defined as the mass of the soil per unit volume.

(i) **Bulk density ( $\rho$ ).** The bulk density or *moist density* is the total mass  $M$  of the soil per unit of its total volume.

Thus,  $\rho = \frac{M}{V} \quad \dots(2.2)$

It is expressed in terms of  $\text{g/cm}^3$  or  $\text{kg/m}^3$ .



(ii) **Dry density** ( $\rho_d$ ). The dry density is the mass of solids per unit of total volume (prior to drying) of the soil mass.

$$\rho_d = \frac{M_d}{V} \quad \dots(2.2 a)$$

(iii) **Density of solids** ( $\rho_s$ ). The density of soil solids is the mass of soil solids ( $M_d$ ) per unit of volume of solids ( $V_s$ ) :

$$\rho_s = \frac{M_d}{V_s} \quad \dots(2.2 b)$$

(iv) **Saturated density** ( $\rho_{sat}$ ). When the soil mass is saturated, its bulk density is called saturated density. Thus, saturated density is the ratio of the total soil mass of saturated sample to its total volume.

(v) **Submerged density** ( $\rho'$ ). The submerged density is the submerged mass of soil solids ( $M_d$ )<sub>sub</sub> per unit of total volume  $V$  of the soil mass :

$$\rho' = \frac{(M_d)_{sub}}{V} \quad \dots(2.2 c)$$

The *submerged density* or *buoyant density* is also expressed as

$$\rho' = \rho_{sat} - \rho_w \quad \dots(2.2 d)$$

where,  $\rho_w$  is the density of water which may be taken as  $1 \text{ g/cm}^3$  for calculation purposes.

(c) **Unit weight of soil mass**. The unit weight of a soil mass is defined as its weight per unit volume.

(i) **Bulk unit weight** ( $\gamma$ ) . The bulk weight or *moist unit weight* is the total weight  $W$  of a soil mass per unit of its total volume  $V$ .

Thus 
$$\gamma = \frac{W}{V} \quad \dots(2.3)$$

(ii) **Dry unit weight** ( $\gamma_d$ ) . The dry unit weight is the weight of solids per unit of its total volume (prior to drying) of the soil mass :

$$\gamma_d = \frac{W_d}{V} \quad \dots(2.4)$$

(iii) **Unit weight of solids** ( $\gamma_s$ ). The unit weight of soil solids is the weight of soil solids  $W_d$  per unit volume of solids ( $V_s$ ) :

$$\gamma_s = \frac{W_d}{V_s} \quad \dots(2.5)$$

Thus, when the dry weight is reckoned with reference to the total original volume  $V$ , it is called the dry unit weight and when it is reckoned with reference to the volume of solids, we get unit weight of soil solids. Since the volume  $V_s$  of the solids does not alter,  $\gamma_s$  is a constant for a given soil, whereas  $\gamma_d$  is not a constant, being dependent upon the initial volume  $V$  of the soil mass.

(iv) **Saturated unit weight ( $\gamma_{sat}$ )**. When the soil mass is saturated, its bulk unit weight is called the saturated unit weight. Thus, saturated unit weight is the ratio of the total weight of a saturated soil sample to its total volume.

(v) **Submerged unit weight ( $\gamma'$ )**. The submerged unit weight  $\gamma'$  is the submerged weight of soil solids  $(W_d)_{sub}$  per unit of total volume  $V$  of the soil mass :

$$\gamma' = \frac{(W_d)_{sub}}{V} \quad \dots(2.6)$$

When the soil mass is submerged, the weight of soil solids is reduced due to buoyancy. The submerged weight  $(W_d)_{sub}$  is, therefore, equal to the weight of soil solids in air minus the weight of water displaced by solids. Hence the submerged unit weight or the *buoyant unit weight* is also expressed as

$$\gamma' = \gamma_{sat} - \gamma_w \quad \dots(2.6 \ a)$$

where,  $\gamma_w$  is the unit weight of water. For calculation purposes in SI units,  $\gamma_w$  may be taken as  $9.81 \text{ kN/m}^3$ .

(d) **Inter-conversion between density and unit weight**. In order to convert the density (expressed in terms of  $\text{g/cm}^3$ ) into unit weight ( $\text{kN/m}^3$ ) multiply the former by 9.81. This is so because

$$1 \text{ g/cm}^3 = \frac{9.81 \times 10^{-6} \text{ kN}}{1 \times 10^{-6} \text{ m}^3} = 9.81 \text{ kN/m}^3$$

$$\text{Hence } \gamma \text{ (kN/m}^3\text{)} = 9.81 \times \rho \text{ (g/cm}^3\text{)}$$

### 2.3. SPECIFIC GRAVITY

Specific gravity  $G$  is defined as the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights being taken in air. In other words, it is the ratio of the unit weight of soil solids to that of water :

$$G = \frac{\gamma_s}{\gamma_w} \quad \dots(2.7)$$

The Indian Standard specifies  $27^\circ \text{C}$  as the *standard temperature* for reporting the specific gravity.

Some qualifying words like : *true, absolute, apparent, bulk* or *mass*, etc., are sometimes added to the term 'specific gravity'. These qualifying words modify the sense of specific gravity as to whether it refers to soil particles or to soil mass. The soil solids have permeable and impermeable voids inside them, the permeable voids being capable of getting filled with water. If all the internal voids of soil particles (permeable and impermeable) are excluded for determining the true volume of solids, the specific gravity obtained is called *absolute* or *true* specific gravity. The *apparent* or *mass* or *bulk* specific gravity  $G_m$  denotes the specific gravity of *soil mass* and is given by



$$G_m = \frac{\gamma}{\gamma_w} \quad \dots(2.8)$$

Unless otherwise specified, we shall denote the Specific Gravity  $G$  (defined by Eq. 2.7) as the specific gravity of soil solids.

Table 2.1 gives the values of specific gravity of some important soil constituents.

TABLE 2.1. SPECIFIC GRAVITY OF SOIL CONSTITUENTS

Soil Constituent	Specific Gravity	Soil Constituent	Specific Gravity
1. Aragonite	2.94	12. Kaolinite	2.64
2. Attapulgite	2.30	13. K. Feldspars	2.54–2.57
3. Augite	3.2–3.4	14. Limonite	3.8
4. Biotite	3.0–3.1	15. Magnetite	5.17
5. Calcite	2.72	16. Montmorillonite	2.75–2.78
6. Chlorite	2.6–2.9	17. Muscovite	2.7–3.1
7. Dolomite	2.85	18. Na-Ca-Feldspars	2.62–2.76
8. Gypsum	2.32	19. Orthoclase	2.56
9. Halloysite (2H <sub>2</sub> O)	2.55	20. Pyrophyllite	2.84
10. Hematite	5.2	21. Quartz	2.65
11. Hornblende	3.2–3.5	22. Serpentine	2.2–2.7
12. Illite	2.6–2.86	23. Talc	2.7

## 2.4. VOIDS RATIO, POROSITY AND DEGREE OF SATURATION

**Voids ratio.** Voids ratio  $e$  of a given soil sample is the ratio of the volume of voids to the volume of soil solids in the given soil mass.

Thus, 
$$e = \frac{V_v}{V_s} \quad \dots(2.9)$$

**Porosity.** The porosity  $n$  of a given soil sample is the ratio of the volume of voids to the *total* volume of the given soil mass

$$n = \frac{V_v}{V} \quad \dots(2.10)$$

The voids ratio  $e$  is generally expressed as a fraction, while the porosity  $n$  is expressed as a percentage and is, therefore, also referred to as *percentage voids*.

Fig. 2.2 (a) shows the soil element in terms of voids ratio  $e$ . If the volume of voids is taken equal to  $e$ , the volume of solids, by definition (Eq. 2.9) would be

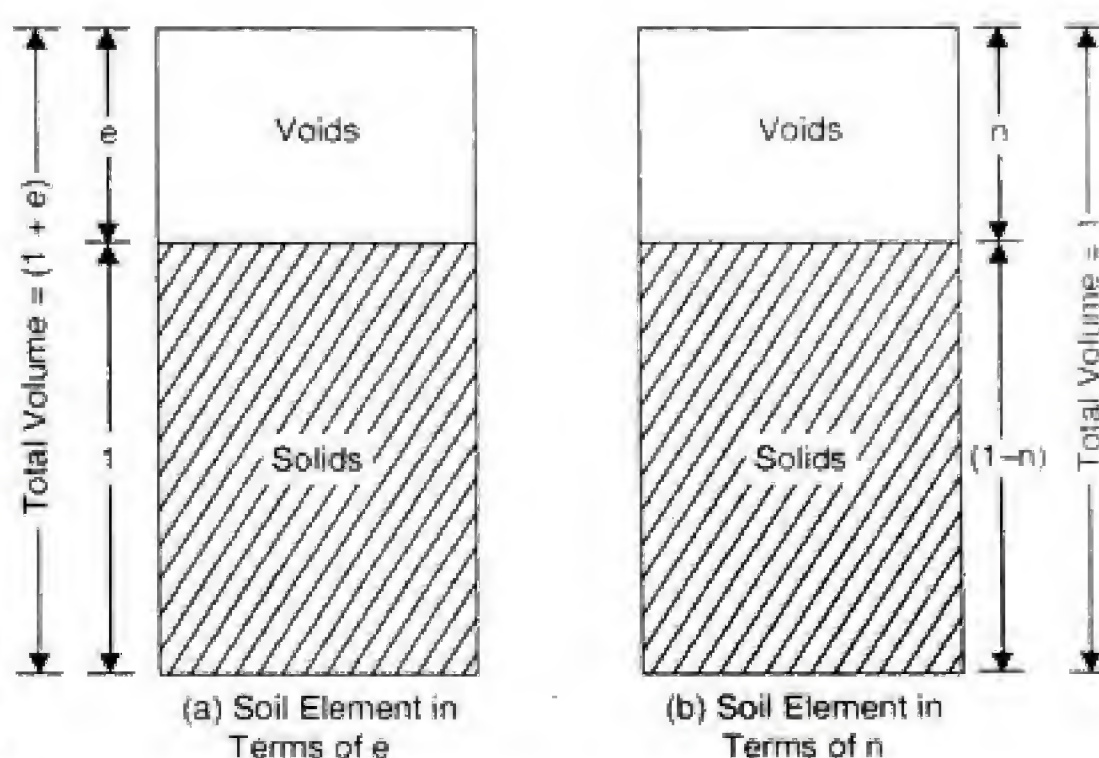


FIG. 2.2. VOIDS RATIO AND POROSITY

equal to 1, and the total volume equal to  $(1 + e)$ . Similarly, if the volume of the voids is taken equal to  $n$  [Fig. 2.2 (b)] the total volume of the element will be 1 (unity) and hence the volume of solids would be equal to  $(1 - n)$ .

From Fig. 2.2 (a), we have by definition of porosity,

$$n = \frac{V_v}{V} = \frac{e}{1 + e} \quad \dots(2.11)$$

Similarly, from Fig. 2.2 (b), we get by definition of voids ratio,

$$e = \frac{V_v}{V_s} = \frac{n}{1 - n} \quad \dots(2.12)$$

Eqs. 2.11 and 2.12 give two relations between  $n$  and  $e$ . Combining Eqs. 2.11 and 2.12, we get

$$n = \frac{e}{1 + e} = e(1 - n)$$

$$\text{or} \quad (1 - n) = \frac{1}{1 + e} \quad \dots(2.12 \text{ a})$$

**Degree of saturation.** In a given volume of voids of a sample, some space is occupied by water and the rest by air. In a *fully saturated* sample, the voids get completely filled with water. The *degree of saturation*  $S$  is defined as the ratio of the volume of water present in a given soil mass to the total volume of voids in it.

$$\text{Thus,} \quad S = \frac{V_w}{V_v} \quad \dots(2.13)$$

The degree of saturation is usually expressed as a percentage and is also known as *percent saturation*. For a fully saturated sample  $V_w = V_v$  and hence  $S = 1$ . For a perfectly dry sample,  $V_w = 0$ , and hence  $S = 0$ .

Depending upon degree of saturation, a soil is generally described as dry, damp, moist etc. Fig. 2.3 shows such *descriptive terms* corresponding to various values of degree of saturation.

**Percentage air voids.** Percentage air voids  $n_a$  is defined as the ratio of the volume of air voids to the total volume of the soil mass and is expressed as percentage :

$$n_a = \frac{V_a}{V} \times 100 \quad \dots(2.14)$$

**Air content.** The air content  $a_c$  is defined as the ratio of volume of air voids to the volume of voids :

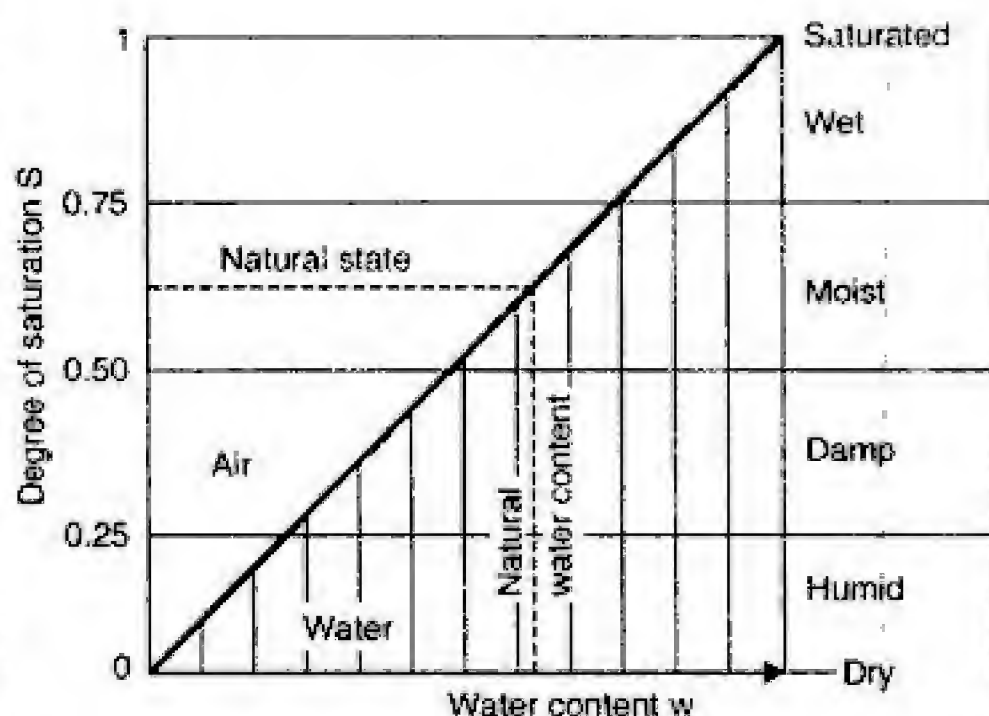


FIG. 2.3 DESCRIPTIVE TERMS OF SOIL STATE WITH VARIOUS VALUES OF  $S$



$$a_c = \frac{V_a}{V_v} \quad \dots(2.15)$$

Since  $V_a = V_v - V_w$ , we get  $a_c = 1 - \frac{V_w}{V_v} = 1 - S$  ...(2.16)

## 2.5. DENSITY INDEX AND RELATIVE COMPACTION

The term *density index*  $I_D$  or *relative density* or *degree of density* is used to express the relative compactness (or degree of compaction) of a natural cohesionless soil deposit. The density index is defined as the ratio of the difference between the voids ratio of the soil in its loosest state  $e_{max}$  and its natural voids ratio  $e$  to the difference between the voids ratios in the loosest and densest states :

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \dots(2.17)$$

where

$e_{max}$  = voids ratio in the loosest state

$e_{min}$  = voids ratio in the densest state

$e$  = natural voids ratio of the deposit.

*This term is used for cohesionless soil only.* This term is not applicable to cohesive soil because of uncertainties in the laboratory determination of the voids ratio in the loosest state of the soil ( $e_{max}$ ). When the natural state of the cohesionless soil is in its loosest form,  $e = e_{max}$  and hence  $I_D = 0$ . When the natural deposit is in its densest state,  $e = e_{min}$  and hence  $I_D = 1$ . For any intermediate state, the density index varies between zero and one.

Equation 2.17, defining density index, can be easily derived by noting the fact that the density index is a function of voids ratio expressed by :

$$I_D = f(e) \quad \dots(1)$$

This relationship between  $I_D$  and  $e$  may be represented graphically, as shown in Fig. 2.4. The slope of the straight line  $AB$ , representing the relationship between  $I_D$  and  $e$  is given by

$$\tan \theta = \frac{1}{e_{max} - e_{min}} \quad \dots(2)$$

or

$$\cot \theta = (e_{max} - e_{min})$$

Now, for an intermediate value  $e$  we have,  $(e_{max} - e) = I_D \cot \theta$

$$I_D = \frac{e_{max} - e}{\cot \theta} \quad \dots(3)$$

Substituting the value of  $\cot \theta$  from Eq. 2, we get

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \dots(2.17)$$

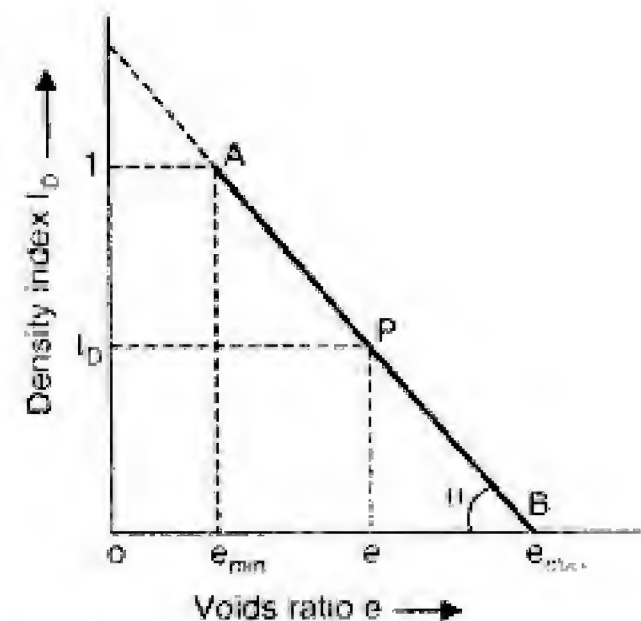


FIG. 2.4.  $I_D - e$  RELATIONSHIP

From Fig. 2.4, we observe that when  $e = e_{\max}$ ,  $I_D = 0$  and when  $e = e_{\min}$ ,  $I_D = 1$ . Now from Eq. 2.24, we have

$$e = \frac{G \gamma_w}{\gamma_d} - 1 \quad ; \quad e_{\max} = \frac{G \gamma_w}{\gamma_{d, \min}} - 1 \quad \text{and} \quad e_{\min} = \frac{G \gamma_w}{\gamma_{d, \max}} - 1.$$

Substituting in Eq. 2.17, we get  $I_D = \frac{\frac{G \gamma_w}{\gamma_{d, \min}} - \frac{G \gamma_w}{\gamma_d}}{\frac{G \gamma_w}{\gamma_{d, \min}} - \frac{G \gamma_w}{\gamma_{d, \max}}}$

or 
$$I_D = \frac{\gamma_d - \gamma_{d, \min}}{\gamma_{d, \max} - \gamma_{d, \min}} \cdot \frac{\gamma_{d, \max}}{\gamma_d} \quad \dots(2.17a)$$

The above equation gives density index in terms of densities.

Density index is also expressed in terms of porosity as follows.

$$I_D = \frac{(n_{\max} - n)(1 - n_{\min})}{(n_{\max} - n_{\min})(1 - n)} \quad \dots(2.17b)$$

where  $\gamma_d$  = in-situ dry density;  $n$  = in-situ porosity.

$\gamma_{d, \max}$  = maximum dry density or dry density corresponding to most compact state

$\gamma_{d, \min}$  = minimum dry density or dry density corresponding to most loosest state

$n_{\max}$  = maximum porosity at loosest state;  $n_{\min}$  = minimum porosity at densest state.

Table 2.2 gives the maximum and minimum voids ratio, porosity and dry unit weight of some typical granular soils. Table 2.3 gives the characteristics of density of granular soils on the basis of relative density.

TABLE 2.2. CHARACTERISTICS OF GRANULAR SOILS IN DENSE AND LOOSE STATES

Description	Voids Ratio		Porosity		Dry unit weight ( $kN/m^3$ )	
	$e_{\max}$	$e_{\min}$	$n_{\max}$	$n_{\min}$	$(\gamma_d)_{\min}$	$(\gamma_d)_{\max}$
1. Uniform spheres	0.92	0.35	47.6	26.0	—	—
2. Clean uniform sand	1.00	0.40	50	29	13	18.5
3. Uniform inorganic silt	1.10	0.40	52	29	12.6	18.5
4. Silty sand	0.90	0.30	47	23	13.6	20.0
5. Fine to coarse sand	0.95	0.20	49	17	13.3	21.7
6. Micaceous sand	1.20	0.40	55	29	12.0	18.8
7. Silty sand and gravel	0.85	0.14	46	12	14.0	23.0

TABLE 2.3. RELATIVE DENSITY

Relative Density (%)	Density Description
0—15	Very loose
15—35	Loose
35—65	Medium
65—85	Dense
85—100	Very dense



**Relative compaction ( $R_C$ )**

*Degree of compaction* is also some times expressed in terms of an *index* called *relative compaction* ( $R_C$ ) defined as follows :

$$R_C = \frac{\gamma_d}{\gamma_{d, \max}} \quad \dots(2.18)$$

where  $\gamma_{d, \max}$  is the maximum dry density from compaction test.

In recent years, the use of the above index has become a generally accepted practice for judging the measure of compaction of both coarse-grained as well as cohesive soils. Since  $\gamma_d = \gamma_s (1 + e)$ , in general, we have

$$R_C = \frac{1 + e_{\min.}}{1 + e} \quad \dots(2.18a)$$

Relative compaction ( $R_C$ ) can also be expressed in terms of relative density ( $I_D$ ) as follows :

$$R_C = \frac{R_0}{1 - I_D (1 - R_0)} \quad \dots(2.18 \ b)$$

where  $R_0 = \gamma_{d, \min} / \gamma_{d, \max}$  and  $R_C$  and  $I_D$  are in fraction form.

Lee and Singh (1971) give the following approximate relation between  $R_C$  and  $I_D$

$$R_C = 80 + 0.2 I_D \quad \dots(2.18c)$$

(where both  $R_C$  and  $I_D$  are in percent form)

When the soil is in looser form,  $I_D = 0$ , which gives minimum value of  $R_C$  as 80% from Eq. 2.18 (c). When the soil is in densest form,  $I_D = 100\%$  corresponding to which  $R_C = 100\%$  from Eq. 2.18 (c). Thus, relative compaction varies from 80% to 100% according to Eq. 2.18 (c).

**2.6. FUNCTIONAL RELATIONSHIPS****(i) Relation between  $e$ ,  $G$ ,  $w$  and  $S$** 

In Fig. 2.5,  $e_w$  represents volume of water,  $e$  represents the volume of voids, and the volume of solids is equal to unity.

$$\text{Now} \quad S = \frac{V_w}{V_v} = \frac{e_w}{e}$$

$$\text{Hence} \quad e_w = e S \quad \dots(2.19 \ a)$$

The term  $e_w$  is known as the *water voids ratio*. For a fully saturated sample,  $e_w = e$ .

$$\text{Now} \quad w = \frac{W_w}{W_d} = \frac{e_w \gamma_w}{\gamma_s \cdot 1}$$

$$\text{But} \quad G = \frac{\gamma_s}{\gamma_w} \quad \text{or} \quad \gamma_s = G \gamma_w$$

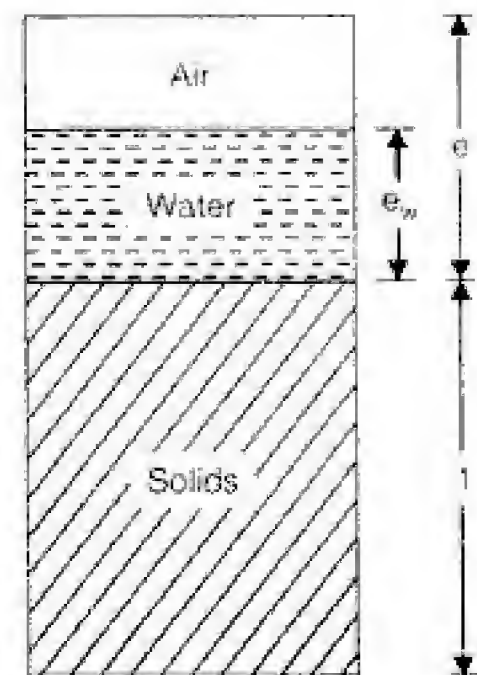


FIG. 2.5. SOIL ELEMENT IN TERMS OF  $e_w$  AND  $e$ .

$$w = \frac{e_w \gamma_w}{G \gamma_w} = \frac{e_w}{G}$$

or  $e_w = w G$  ... (2.19 b)

Equating Eqs. 2.19 (a) and 2.19 (b), we get

$$e = \frac{w G}{S} \quad \dots (2.20)$$

For a fully saturated soil,  $S = 1$  and  $w = w_{sat}$

$$e = w_{sat} G \quad \dots (2.20 a)$$

(ii) Relation between  $e$ ,  $S$  and  $n_a$

$$n_a = \frac{V_a}{V}$$

From Fig. 2.5,  $V_a = V_v - V_w = e - e_w$  and  $V = V_s + V_v = 1 + e$

$$\therefore n_a = \frac{e - e_w}{1 + e} \quad \text{But } e_w = e S \text{ (Eq. 2.18)}$$

$$\therefore n_a = \frac{e (1 - S)}{1 + e} \quad \dots (2.21)$$

(iii) Relation between  $n_a$ ,  $a_c$  and  $n$

$$a_c = \frac{V_a}{V_v}, \quad n = \frac{V_v}{V}$$

$$\therefore n_a = \frac{V_a}{V} = n \cdot a_c \quad \dots (2.22)$$

(iv) Relation between  $\gamma_d$ ,  $G$  and  $e$  (or  $n$ )

$$\gamma_d = \frac{W_d}{V} \quad \gamma_d = \frac{\gamma_s \cdot V_s}{V} \quad \dots (i)$$

Now from Fig. 2.2 (a),

$$V_s = 1 \text{ and } V = (1 + e)$$

$$\therefore \gamma_d = \frac{\gamma_s \cdot 1}{1 + e} \quad \text{But } \gamma_s = G \gamma_w$$

$$\therefore \gamma_d = \frac{G \gamma_w}{1 + e} \quad \dots (2.23)$$

A convenient expression for calculating the voids ratio of soil mass is obtained from Eq. 2.23, as

$$e = \frac{G \gamma_w}{\gamma_d} - 1 \quad \dots (2.24)$$

Again, from Fig. 2.2 (b),  $V_s = (1 - n)$  and  $V = 1$

Substituting in (i) and taking  $\gamma_s = G \gamma_w$  we get

$$\gamma_d = \frac{G \gamma_w (1 - n)}{1} = (1 - n) G \gamma_w \quad \dots (2.25)$$



**(v) Relation between  $\gamma_{sat}$ ,  $G$  and  $e$  (or  $n$ )**

$$\gamma_{sat} = \frac{\text{Total weight of saturated soil}}{\text{Total volume of soil}} = \frac{W_{sat}}{V} = \frac{W_d + W_w}{V} = \frac{\gamma_s V_s + \gamma_w V_w}{V} \quad \dots(i)$$

From Fig. 2.2 (a),  $V_s = 1$ ,  $V_w = e$  and  $V = 1 + e$

$$\therefore \gamma_{sat} = \frac{\gamma_s \cdot 1 + \gamma_w \cdot e}{1 + e} = \frac{G \gamma_w + \gamma_w \cdot e}{1 + e}$$

$$\text{or} \quad \gamma_{sat} = \frac{(G + e) \gamma_w}{1 + e} \quad \dots(2.26)$$

Similarly, from Fig. 2.2 (b),  $V_s = 1 - n$ ;  $V_w = n$ ;  $V = 1$

$$\therefore \gamma_{sat} = \frac{\gamma_s (1 - n) + \gamma_w n}{1}$$

$$\text{or} \quad \gamma_{sat} = G \gamma_w (1 - n) + \gamma_w n \quad \dots(2.27)$$

**(vi) Relation between  $\gamma$ ,  $G$ ,  $e$ , and  $S$** 

$$\text{We have, by definition,} \quad \gamma = \frac{W}{V} = \frac{\gamma_s V_s + \gamma_w V_w}{V}$$

From Fig. 2.4, illustrating a partially saturated sample, we have

$$V_s = 1; V_w = e_w \text{ and } V = (1 + e)$$

$$\therefore \gamma = \frac{\gamma_s \cdot 1 + \gamma_w \cdot e_w}{1 + e} \quad \text{But } \gamma_s = G \gamma_w \text{ and } e_w = e S$$

$$\therefore \gamma = \frac{G \gamma_w + \gamma_w \cdot e S}{1 + e} = \frac{(G + e S) \gamma_w}{1 + e} \quad \dots(2.28)$$

This is a general relationship from which Eqs. 2.23 and 2.26 can be derived. For example, when the soil is perfectly dry,  $S = 0$  and hence  $\gamma$  reduces to

$$\gamma_d = \frac{G \gamma_w}{1 + e} \quad \text{Eq. (2.23)}$$

Similarly, when  $S = 1$ ,  $\gamma$  becomes

$$\gamma_{sat} = \frac{(G + e) \gamma_w}{1 + e} \quad \text{Eq. (2.26)}$$

**(vii) Relation between  $\gamma'$ ,  $G$  and  $e$** 

$$\gamma' = \gamma_{sat} - \gamma_w = \frac{(G + e) \gamma_w}{1 + e} - \gamma_w \quad \text{or} \quad \gamma' = \frac{(G - 1) \gamma_w}{1 + e} \quad \dots(2.29)$$

**(viii) Relation between  $\gamma_d$ ,  $\gamma$  and  $w$** 

$$\text{Water content } w = \frac{W_w}{W_d} \quad \text{Hence } 1 + w = \frac{W_w + W_d}{W_d} = \frac{W}{W_d}$$

$$\therefore W_d = \frac{W}{1 + w} \quad \dots(2.30)$$

Now 
$$\gamma_d = \frac{W_d}{V} = \frac{W}{(1+w)V} \quad \therefore \quad \gamma_d = \frac{\gamma}{1+w} \quad \dots(2.31)$$

(ix) **Relation between  $\gamma'$ ,  $\gamma_d$  and  $n$**  : From Fig. 2.3,

$$(W_d)_{sub} = 1 \cdot \gamma_s - 1 \cdot \gamma_w = G \gamma_w - \gamma_w$$

or 
$$(W_d)_{sub} = (G - 1)\gamma_w \quad \text{and} \quad V = 1 + e$$

$$\therefore \quad \gamma' = \frac{(W_d)_{sub}}{V} = \frac{(G - 1)\gamma_w}{1 + e} \quad (\text{Eq. 2.29}) \quad \text{or} \quad \gamma' = \frac{G\gamma_w}{1 + e} - \frac{\gamma_w}{1 + e}$$

But 
$$\frac{G\gamma_w}{1 + e} = \gamma_d \quad (\text{Eq. 2.23}) \quad \text{and} \quad \frac{1}{1 + e} = 1 - n \quad [\text{Eq. 2.12 (a)}]$$

$$\gamma' = \gamma_d - (1 - n)\gamma_w \quad \dots(2.32)$$

(x) **Relation between  $\gamma_{sat}$ ,  $\gamma$ ,  $\gamma_d$  and  $S$**  : We have already proved in Eq. 2.28,

that 
$$\gamma = \frac{(G + eS)\gamma_w}{1 + e} \quad \text{or} \quad \gamma = \frac{G\gamma_w}{1 + e} + S \frac{e\gamma_w}{1 + e}$$

or 
$$\gamma = \gamma_d + S \left[ \frac{(G + e)\gamma_w}{1 + e} - \frac{G\gamma_w}{1 + e} \right]$$

Hence 
$$\gamma = \gamma_d + S[\gamma_{sat} - \gamma_d] \quad \dots(2.33)$$

(xi) **Relation between  $\gamma_d$ ,  $G$ ,  $w$  and  $S$**

From Eq. 2.23, 
$$\gamma_d = \frac{G\gamma_w}{1 + e} \quad \text{But} \quad e = \frac{wG}{S} \quad (\text{from Eq. 2.20})$$

$$\therefore \quad \gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{S}} \quad \dots(2.34)$$

When 
$$S = 1, \quad \gamma_d = \frac{G\gamma_w}{1 + w_{sat}G} \quad \dots(2.34 \text{ a})$$

(xii) **Relation between  $\gamma_d$ ,  $G$ ,  $w$  and  $n_a$**

From Fig. 2.1, 
$$V = V_a + V_w + V_s \quad \text{or} \quad V = V_a + \frac{W_w}{\gamma_w} + \frac{W_d}{\gamma_s}$$

$$\therefore \quad 1 = \frac{V_a}{V} + \frac{wW_d}{\gamma_w V} + \frac{W_d}{\gamma_s V} = \frac{V_a}{V} + \frac{w\gamma_d}{\gamma_w} + \frac{\gamma_d}{\gamma_s}$$

$$\therefore \quad \left(1 - \frac{V_a}{V}\right) = \frac{w\gamma_d}{\gamma_w} + \frac{\gamma_d}{G\gamma_w} \quad \text{or} \quad (1 - n_a) = \frac{\gamma_d}{\gamma_w} \left(w + \frac{1}{G}\right)$$

$$\therefore \quad \gamma_d = \frac{(1 - n_a)\gamma_w}{w + \frac{1}{G}} \quad \text{or} \quad \gamma_d = \frac{(1 - n_a)G\gamma_w}{1 + wG} \quad \dots(2.35)$$

**Inter-relationships in terms of densities** : Similar inter-relationships can be obtained in terms of densities ( $\rho$ ). Thus, we have the following inter-relationships :



$$\rho_d = \frac{G \cdot \rho_w}{1 + e} \quad \dots(2.23 \text{ a}) ; \quad e = \frac{G \cdot \rho_w}{\rho_d} - 1 \quad \dots(2.24 \text{ a})$$

$$\rho_d = (1 - n) G \rho_w \quad \dots(2.25 \text{ a}) ; \quad \rho_{sat} = \frac{(G + e) \rho_w}{1 + e} \quad \dots(2.26 \text{ a})$$

$$\rho_{sat} = G \cdot \rho_w (1 - n) + \rho_w \cdot n \quad \dots(2.27 \text{ a}) ; \quad \rho = \frac{(G + e S) \rho_w}{1 + e} \quad \dots(2.28 \text{ a})$$

$$\rho' = \frac{(G - 1) \rho_w}{1 + e} \quad \dots(2.29 \text{ a}) ; \quad \rho_d = \frac{\rho}{1 + w} \quad \dots(2.31 \text{ a})$$

$$\rho' = \rho_d - (1 - n) \rho_w \quad \dots(2.32 \text{ a}) ; \quad \rho = \rho_d + S (\rho_{sat} - \rho_d) \quad \dots(2.33 \text{ a})$$

$$\rho_d = \frac{G \rho_w}{1 + \frac{w G}{S}} \quad \dots(2.34 \text{ a}) ; \quad \text{and } \rho_d = \frac{(1 - n_a) G \rho_w}{1 + w G} \quad \dots(2.35 \text{ a})$$

**Example 2.1.** A soil sample has a porosity of 40 per cent. The specific gravity of solids is 2.70. Calculate (a) voids ratio, (b) dry density, (c) unit weight if the soil is 50% saturated and (d) unit weight if the soil is completely saturated.

**Solution :** Given,  $n = 40 \% = 0.4$  ;  $G = 2.70$

(a) We have  $e = \frac{n}{1 - n} = \frac{0.4}{1 - 0.4} = 0.667$

(b)  $\gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.7 \times 9.81}{1 + 0.667} = 15.89 \text{ kN/m}^3$  (Taking  $\gamma_w = 9.81 \text{ kN/m}^3$ )

(c)  $e = \frac{w G}{S}$  or  $w = \frac{e S}{G} = \frac{0.667 \times 0.5}{2.70} = 0.124$

$\gamma_d = 15.89 \text{ kN/m}^3$  (as before)

$\therefore \gamma = \gamma_d (1 + w) = 15.89 \times 1.124 = 17.85 \text{ kN/m}^3$

**Check :**  $\gamma = \frac{\gamma_w (G + e S)}{1 + e} = \frac{9.81 (2.70 + 0.667 \times 0.5)}{1 + 0.667} = 17.85 \text{ kN/m}^3$

(d) When the soil is fully saturated,  $e = w_{sat} \cdot G$

$\therefore w_{sat} = \frac{e}{G} = \frac{0.667}{2.70} = 0.247$

$\therefore \gamma_{sat} = \gamma_d (1 + w_{sat}) = 15.89 \times 1.247 = 19.81 \text{ kN/m}^3$

Alternatively,  $\gamma_{sat} = G \gamma_w (1 - n) + \gamma_w n = 2.7 \times 9.81 (1 - 0.4) + 9.81 \times 0.4$   
 $= 15.89 + 3.92 = 19.81 \text{ kN/m}^3$

**Example 2.2.** An undisturbed sample of soil has a volume of  $100 \text{ cm}^3$  and mass of  $190 \text{ g}$ . On oven drying for 24 hours, the mass is reduced to  $160 \text{ g}$ . If the specific gravity of grains is 2.68, determine the water content, voids ratio and degree of saturation of the soil.

**Solution :**  $M_w = 190 - 160 = 30 \text{ g}$  ;  $M_d = 160 \text{ g}$

$$\therefore w = \frac{M_w}{M_d} = \frac{30}{160} = 0.188 = 18.8 \%$$

$$\text{Mass of moist soil} = M = 190 \text{ g} \quad \therefore \text{Bulk density } \rho = \frac{M}{V} = \frac{190}{100} = 1.9 \text{ g/cm}^3$$

$$\text{Hence } \gamma = 9.81 \times \rho = 9.81 \times 1.9 = 18.64 \text{ kN/m}^3$$

$$\text{Alternatively, } \gamma = \frac{W}{V} = \frac{190 \text{ (g)}}{100 \text{ (cm}^3\text{)}} \times 9.81 = 18.64 \text{ kN/m}^3$$

$$(\text{Since } 1 \text{ g/cm}^3 = 9.81 \text{ kN/m}^3)$$

$$\therefore \gamma_d = \frac{\gamma}{1 + w} = \frac{18.64}{1 + 0.188} = 15.69 \text{ kN/m}^3$$

$$\text{Alternatively, } \gamma_d = \frac{W_d}{V} = \frac{160}{100} \times 9.81 = 15.69 \text{ kN/m}^3$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 9.81}{15.69} - 1 = 0.67$$

$$S = \frac{w G}{e} = \frac{0.188 \times 2.68}{0.67} = 0.744 = 74.4 \%$$

**Example 2.3.** The in-situ density of an embankment, compacted at a water content of 12% was determined with the help of a core cutter. The empty mass of the cutter was 1286 g and the cutter full of soil had a mass of 3195 g, the volume of the cutter being 1000 cm<sup>3</sup>. Determine the bulk density, dry density and the degree of saturation of the embankment.

If the embankment becomes fully saturated during rains, what would be its water content and saturated unit weight? Assume no volume change in soil on saturation. Take the specific gravity of the soil as 2.70.

**Solution.** Mass of soil in cutter  $M = 3195 - 1286 = 1909 \text{ g}$

$$\therefore \text{Bulk density } \rho = \frac{M}{V} = \frac{1909}{1000} = 1.909 \text{ g/cm}^3$$

$$\therefore \text{Bulk unit weight } \gamma = 9.81 \rho = 9.81 \times 1.909 = 18.73 \text{ kN/m}^3$$

$$\gamma_d = \frac{\gamma}{1 + w} = \frac{18.73}{1 + 0.12} = 16.72 \text{ kN/m}^3$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.72} - 1 = 0.584$$

$$\text{and } S = \frac{w G}{e} = \frac{0.12 \times 2.70}{0.584} = 0.555 = 55.5 \%$$

**At Saturation.** Since the volume remains the same, the voids ratio also remains unchanged. Now  $e = w_{sat} \cdot G$

$$\therefore w_{sat} = \frac{e}{G} = \frac{0.584}{2.70} = 0.216 = 21.6 \%$$



$$\gamma_{sat} = \left( \frac{G + e}{1 + e} \right) \gamma_w = \frac{2.7 + 0.584}{1 + 0.584} \times 9.81 = 20.34 \text{ kN/m}^3$$

**Example 2.4.** The in-situ percentage voids of a sand deposit is 34 per cent. For determining the density index, dried sand from the stratum was first filled loosely in a  $1000 \text{ cm}^3$  mould and was then vibrated to give a maximum density. The loose dry mass in the mould was 1610 g and the dense dry mass at maximum compaction was found to be 1980 g. Determine the density index if the specific gravity of the sand particles is 2.67.

**Solution.**  $n = 34 \% = 0.34$

$$e = \frac{n}{1 - n} = \frac{0.34}{1 - 0.34} = \frac{0.34}{0.66} = 0.515 \quad ; \quad \gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.67 \times 9.81}{1 + 0.515} = 17.289$$

$$(\gamma_d)_{max} = \frac{1980}{1000} \times 9.81 = 19.42 \text{ kN/m}^3 \quad ; \quad (\gamma_d)_{min} = \frac{1610}{1000} \times 9.81 = 15.79 \text{ kN/m}^3$$

$$e_{min} = \frac{G \gamma_w}{(\gamma_d)_{max}} - 1 = \frac{2.67 \times 9.81}{19.42} - 1 = 0.349 \quad ; \quad e_{max} = \frac{G \gamma_w}{(\gamma_d)_{min}} - 1 = \frac{2.67 \times 9.81}{15.79} - 1 = 0.659$$

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.659 - 0.515}{0.659 - 0.349} = 0.465 = 46.5 \%$$

**Example 2.5.** The mass specific gravity (apparent specific gravity) of a soil equals 1.64. The specific gravity of solids is 2.70. Determine the voids ratio under the assumption that the soil is perfectly dry. What would be the voids ratio, if the sample is assumed to have a water content of 8 per cent ?

**Solution :** When the sample is dry,  $G_m = \frac{\gamma_d}{\gamma_w} = 1.64$  (given)

$$\therefore \gamma_d = 1.64 \gamma_w = 1.64 \times 9.81 = 16.09 \text{ kN/m}^3$$

$$\text{Now} \quad e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.09} - 1 = 0.646.$$

When the sample has the water content  $w = 8\%$ ,  $G_m = \frac{\gamma}{\gamma_w} = 1.64$

$$\therefore \gamma = 1.64 \gamma_w = 1.64 \times 9.81 = 16.09 \text{ kN/m}^3$$

$$\therefore \gamma_d = \frac{\gamma}{1 + w} = \frac{16.09}{1 + 0.08} = 14.9 \text{ kN/m}^3$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{14.9} - 1 = 0.78.$$

**Example 2.6.** A natural soil deposit has a bulk unit weight of  $18.44 \text{ kN/m}^3$  and water content of 5 per cent. Calculate the amount of water required to be added to 1 cubic metre of soil to raise the water content to 15 per cent. Assume the voids ratio to remain constant. What will then be the degree of saturation ? Assume  $G = 2.67$ .

**Solution :** Given :  $\gamma = 18.44 \text{ kN/m}^3$  and  $w = 5 \%$

$$\therefore \gamma_d = \frac{\gamma}{1 + w} = \frac{18.44}{1 + 0.05} = 17.56 \text{ kN/m}^3$$

Earlier, when  $w = 5\%$ ,  $w = 0.05 = \frac{W_w}{W_d}$ . For one cubic metre of soil  $V = 1 \text{ m}^3$

$$\therefore W_d = \gamma_d \cdot V = 17.56 \times 1 = 17.56 \text{ kN}$$

$$\therefore W_w = 0.05 \times W_d = 0.05 \times 17.56 = 0.878 \text{ kN}$$

$$\therefore V_w = \frac{W_w}{\gamma_w} = \frac{0.878}{9.81} = 0.0895 \text{ m}^3$$

Later, when  $w = 15\%$  ;  $W_w = w W_d = 0.15 \times 17.56 = 2.634 \text{ kN}$

$$\therefore V_w = \frac{W_w}{\gamma_w} = \frac{2.634}{9.81} = 0.2685 \text{ m}^3$$

Hence additional water required to raise the water content from 5% to 15%  
 $= 0.2685 - 0.0895 = 0.179 \text{ m}^3 = 179 \text{ litres}$ .

$$\text{Voids ratio, } e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{17.56} - 1 = 0.49$$

After the water has been added,  $e$  remains the same.

$$\therefore S = \frac{w G}{e} = \frac{0.15 \times 2.67}{0.49} = 0.817 = 81.7\%.$$

**Example 2.7.** Calculate the unit weights and specific gravities of solids of (a) a soil composed of pure quartz and (b) a soil composed of 60% quartz, 25% mica and 15% iron oxide. Assume that both soils are saturated and have voids ratio of 0.63. Take average  $G$  for quartz = 2.66, for mica = 3.0 and for iron oxide = 3.8.

**Solution :**

(a) For the soil composed of pure quartz,  $G$  for quartz = 2.66

$$\therefore \gamma_{sat} = \frac{G + e}{1 + e} \gamma_w = \frac{2.66 + 0.63}{1 + 0.63} \times 9.81 = 19.8 \text{ kN/m}^3$$

(b) For the composite soil,

$$G_{average} = (2.66 \times 0.6) + (3.0 \times 0.25) + (3.8 \times 0.15) = 1.60 + 0.75 + 0.57 = 2.92$$

$$\gamma_{sat} = \frac{2.92 + 0.63}{1 + 0.63} \times 9.81 = 21.36 \text{ kN/m}^3$$

**Example 2.8.** A soil has a bulk unit weight of  $20.11 \text{ kN/m}^3$  and water content of 15 per cent. Calculate the water content if the soil partially dries to a unit weight of  $19.42 \text{ kN/m}^3$  and the voids ratio remains unchanged.

**Solution :** Before drying,  $\gamma = 20.11 \text{ kN/m}^3$

$$\therefore \gamma_d = \frac{20.11}{1 + 0.15} = 17.49 \text{ kN/m}^3$$

Since, after drying,  $e$  does not change,  $V$  and  $\gamma_d$  are the same,



$$\therefore \gamma = \gamma_d (1 + w) \quad \text{or} \quad 1 + w = \frac{\gamma}{\gamma_d} = \frac{19.42}{17.49} = 1.11$$

$$\therefore w = 1.11 - 1 = 0.11 = 11 \%$$

**Example 2.9.** A cube of dried clay having sides 4 cm long has a mass of 110 g. The same cube of soil, when saturated at unchanged volume, has mass of 135 g. Draw the soil element showing the volumes and weights of the constituents, and then determine the specific gravity of soil solids and the voids ratio.

**Solution :** Volume of soil =  $4^3 = 64 \text{ cm}^3$  (Before and after saturation).

$$\text{Mass of water after saturation} = 135 - 110 = 25 \text{ g}$$

$$\therefore \text{Volume of voids} = 25 \text{ cm}^3$$

$$\text{Volume of solids } V_s = 64 - 25 = 39 \text{ cm}^3$$

$$\text{Total mass of dry solids } M_s = 110 \text{ g}$$

The mass and volume has been marked in Fig. 2.6, from

$$\text{which : } G = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w} = \frac{M_s}{V_s \times 1} = \frac{110}{39 \times 1} = 2.82$$

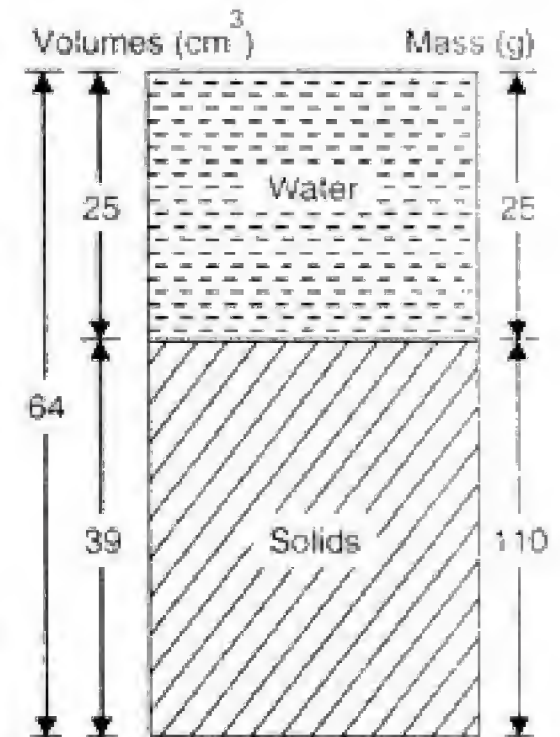


FIG. 2.6

$$e = \frac{V_v}{V_s} = \frac{25}{39} = 0.64$$

**Example 2.10.** It is required to prepare a compacted cylindrical specimen of 40 mm dia. and 80 mm length from oven dry soil. The specimen is required to have water content of 10% and percent air voids of 18%. Taking  $G = 2.70$ , determine the mass of soil and mass of water, required for the preparation of the above specimen.

**Solution**

$$\text{Total volume of compacted sample} = \frac{\pi}{4} (0.04)^2 \times 0.08 = 1.0053 \times 10^{-4} \text{ m}^3 \quad \dots(1)$$

Let the mass of solids (i.e. dry weight of soil) =  $M_d$  kg.

$$\therefore \text{Mass of water, } M_w = wM_d = 0.16 M_d$$

$$\text{Volume of solids, } V_s = \frac{M_d}{G \gamma_w} = \frac{M_d}{2.7 \times 1000} = \frac{M_d}{2700} \text{ m}^3 \quad \dots(2)$$

$$\text{Volume of water, } V_w = \frac{M_w}{\rho_w} = \frac{0.16 M_d}{1000} = 1.6 \times 10^{-4} M_d \text{ m}^3 \quad \dots(3)$$

$$\text{Volume of air, } V_a = n_a \cdot V = 0.18 \times 1.0053 \times 10^{-4} = 1.8095 \times 10^{-5} \text{ m}^3 \quad \dots(4)$$

$$\text{Now, total volume } V = V_s + V_w + V_a$$

$$\therefore 1.0053 \times 10^{-4} = \frac{M_d}{2700} + 1.6 \times 10^{-4} M_d + 1.8095 \times 10^{-5}$$

$$\text{From which, we get } M_d = 0.1554 \text{ kg} = 155.4 \text{ g}$$

Mass of water,  $M_w = 0.16 M_d = 0.16 \times 0.1554 = 0.0249 \text{ kg} = 24.9 \text{ g}$

**Example 2.11.** An embankment, having a total volume of  $5000 \text{ m}^3$  has a water content of 16% and dry density of  $1.75 \text{ g/cm}^3$ . If it was constructed from a borrow pit where the undisturbed soil has a water content of 13% and voids ratio of 0.6, calculate the quantity of soil which was excavated for the construction of the above embankment. Take specific gravity of soil solids as 2.68.

**Solution :** Total volume of embankment,  $V = 5 \times 10^4 \text{ m}^3$

Now dry density  $\rho_d = \frac{M_d}{V} = 1.75 \text{ g/cm}^3 = 1.75 \text{ t/m}^3$

$\therefore M_d = \rho_d \cdot V = 1.75 \times 5 \times 10^4 = 87500 \text{ t}$

In the borrow pit,  $(\rho_d)_b = \frac{G \rho_w}{1 + e} = \frac{2.68 \times 1}{1 + 0.6} = 1.675 \text{ t/m}^3$

Now, volume of soil from borrow pit,  $V_b = \left( \frac{M_d}{\rho_d} \right)_b = \frac{87500}{1.675} = 52239 \text{ m}^3$

**Example 2.12.** An embankment, having total volume of  $2000 \text{ m}^3$  is to be constructed having a bulk density of  $1.98 \text{ g/cm}^3$  and a placement water content of 18%. The soil is to be obtained either from borrow area A or borrow area B, which have voids ratio of 0.78 and 0.69 respectively and water content of 16% and 12% respectively. Taking  $G = 2.66$  for both the soils, determine the volume of soil required to be excavated from each of the areas. If the cost of excavation is Rs. 35 per  $\text{m}^3$  in each area, but cost of transportation is Rs. 32 and 36 per  $\text{m}^3$  from areas A and B respectively, which of the borrow area is more economical ?

**Solution:** For the embankment,  $\rho_d = \frac{\rho}{1 + w} = \frac{1.98}{1 + 0.18} = 1.678 \text{ g/cm}^3 = 1.678 \text{ t/m}^3$

$\therefore M_d = \rho_d \cdot V = 1.678 \times 2000 = 3356 \text{ t}$

(a) Borrow area A :  $\rho_d = \frac{G \rho_w}{1 + e} = \frac{2.66 \times 1}{1 + 0.78} = 1.494 \text{ g/cm}^3 = 1.494 \text{ t/m}^3$

$\therefore V = \frac{M_d}{\rho_d} = \frac{3356}{1.494} = 2246 \text{ m}^3$

Cost of excavation and transportation =  $(35 + 32) 2246 = \text{Rs. } 150482$

(b) Borrow area B :  $\rho_d = \frac{G \rho_w}{1 + e} = \frac{2.66 \times 1}{1 + 0.69} = 1.574 \text{ g/cm}^3 = 1.574 \text{ t/m}^3$

$\therefore V = \frac{M_d}{\rho_d} = \frac{3356}{1.574} = 2132 \text{ m}^3$

Cost of excavation and transportation =  $(35 + 36) 2132 = \text{Rs. } 151372$

Hence borrow area A is more economical.



**Example 2.13.** *Sandy soil in a borrow pit has unit weight of solids as  $26.3 \text{ kN/m}^3$ , water content equal to 11% and bulk unit weight equal to  $16.4 \text{ kN/m}^3$ . How many cubic meter of compacted fill could be constructed of  $3500 \text{ m}^3$  of sand excavated from the borrow pit, if the required value of porosity in the compacted fill is 30%. Also compute the change in degree of saturation.*

**Solution :** Let us use suffix 1 for the borrow pit soil and 2 for the compacted soil. Assuming that weight and water content do not change during construction, the change in the volume can be calculated from the change in the unit weight.

$$\text{i.e.} \quad \frac{V_1}{V_2} = \frac{W/\gamma_1}{W/\gamma_2} = \frac{\gamma_2}{\gamma_1} = \frac{1+e_1}{1+e_2}$$

$$\text{Hence} \quad V_2 = V_1 \left( \frac{1+e_2}{1+e_1} \right) = V_1 \left( \frac{1-n_1}{1-n_2} \right) \quad \dots(1)$$

$$\text{From Eq. 2.24,} \quad e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{\gamma_s}{\gamma/(1+w)} - 1 = \frac{\gamma_s (1+w)}{\gamma} - 1$$

$$\text{Hence} \quad e_1 = \frac{\gamma_s (1+w)}{\gamma_1} - 1 = \frac{26.3 (1+0.11)}{16.4} - 1 = 0.780$$

$$\text{Also,} \quad e_2 = \frac{n_2}{1-n_2} = \frac{0.3}{1-0.3} = 0.429$$

$$\text{Hence} \quad V_2 = V_1 \left( \frac{1+e_2}{1+e_1} \right) = 3500 \left( \frac{1+0.429}{1+0.780} \right) = 2810 \text{ m}^3$$

$$\text{Now} \quad S_1 = \frac{w \gamma_s}{e_1 \gamma_w} = \frac{0.11 \times 26.3}{0.78 \times 9.81} = 0.378$$

$$S_2 = \frac{w \gamma_s}{e_2 \gamma_w} = \frac{0.11 \times 26.3}{0.429 \times 9.81} = 0.687$$

Thus, the degree of saturation will be considerably increased in the compacted fill.

**Example 2.14. :** *A sand deposit has following properties in its natural state :*

- (i) *bulk unit weight :  $18.6 \text{ kN/m}^3$*       (ii) *Unit weight of soil solids :  $26.3 \text{ kN/m}^3$*   
 (iii) *water content : 10%*

*The same sand, when dumped loosely will experience a volume change of 18% due to bulking. Determine the porosity of the sand in both the states.*

$$\text{Now} \quad \gamma_1 = \gamma_s \frac{1+w}{1+e_1} = \gamma_s (1+w) (1-n_1)$$

$$\therefore \quad 1-n = \frac{\gamma_1}{\gamma_s (1+w)} = \frac{18.6}{26.3 (1+0.1)} = 0.643$$

$$\text{or} \quad n_1 = 1 - 0.643 = 0.357 = 35.7\%$$

Let us now assume that weight and water content of the material do not change during dumping.

$$\therefore \frac{V_1}{V_2} = \frac{W/\gamma_1}{W/\gamma_2} = \frac{\gamma_2}{\gamma_1} = \frac{1+e_1}{1+e_2} = \frac{1-n_2}{1-n_1}$$

$$\text{or} \quad 1-n_2 = \frac{V_1}{V_2} (1-n_1)$$

$$\text{or} \quad n_2 = 1 - \frac{V_1}{V_2} (1-n_1) = 1 - \frac{1}{1.18} (1-0.357) = 0.455 = \mathbf{45.5\%}$$

## 2.7. PACKING OF UNIFORM SPHERES

Packing of *uniform spheres* is the only granular system, the voids ratio and porosity of which can be determined mathematically. According to Slichter (1899) the loosest stable arrangement of equal sized spheres is obtained when the sphere centres form a rectangular space lattice. Such a packing, shown in Fig. 2.7 (a) is known as *cubic packing*, wherein each sphere is in contact with six neighboring spheres. The porosity for such a packing is  $n = 47.64\%$  (see example 2.15).

Similarly, the densest state of packing is obtained (Fig. 2.7 b) when sphere centres form a rhombohedral array ( $\alpha = 60^\circ$ ) wherein each sphere is in contact with 12 neighboring spheres. The porosity for such a packing is  $n = 25.95\%$  (see example 2.16).

For an *intermediate packing* (i.e. neither loosest packing nor densest packing), the centres of any 8 spheres, originally arranged in cubic packing form the corners of a rhombohedron, with acute face angle  $\alpha$ , known as angle of *orientation of the rhombohedron*. Corresponding to original unit cube, the total volume of the rhombohedron, or volume of *unit cell* is given by.

$$V = \sqrt{1 + 2 \cos \alpha} (1 - \cos \alpha) \quad \dots(2.36)$$

The volume of solids in this rhombohedron is evidently equal to original volume  $\pi/6$ , which is constant and which does not change with the angle  $\alpha$ . The porosity for such an intermediate packing is given by the following formula by Slichter :

$$n = 1 - \frac{\pi}{6 (1 - \cos \alpha) \sqrt{1 + 2 \cos \alpha}} \quad \dots(2.37)$$

**Example 2.15.** Determine the maximum possible voids ratio for a uniformly graded sand of perfectly spherical grains.

**Solution:** The soil will have maximum possible voids when its grains are arranged in a cubical array of spheres. Consider a *unit cube* of soil having spherical particles of diameter  $d$ .

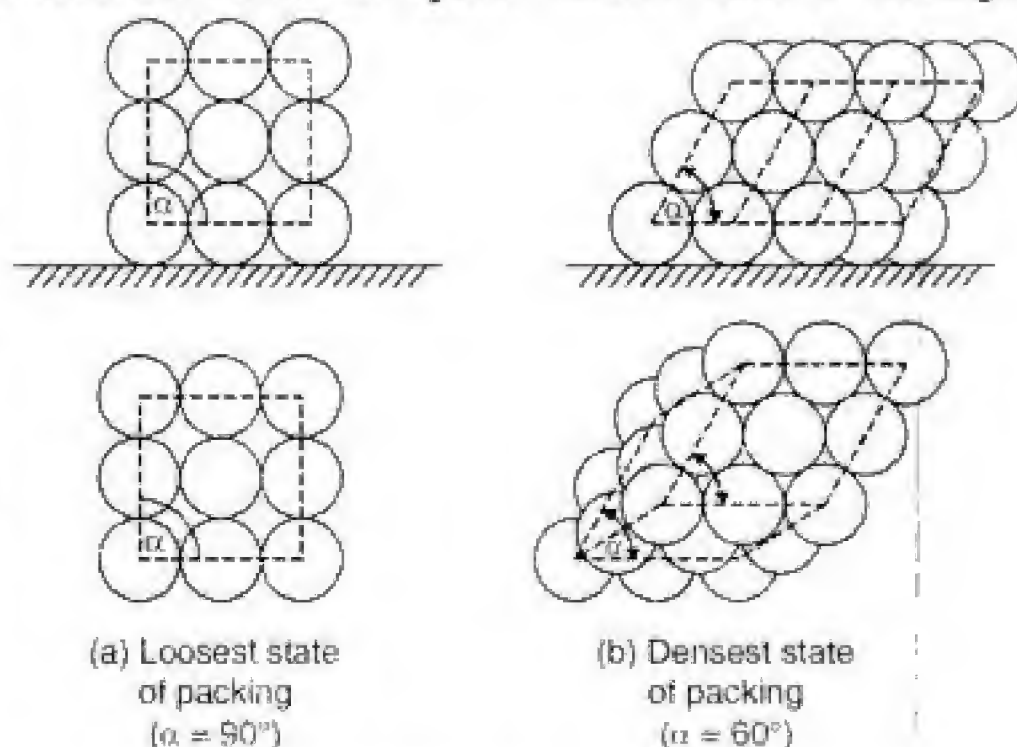


FIG. 2.7. PACKING OF UNIFORM SPHERES.



$$\text{Volume of each spherical particle} = \frac{\pi}{6} d^3$$

$$\text{Total volume of container} = 1 \times 1 \times 1 = 1$$

$$\therefore \text{No. of solids in the container} = \frac{1}{d} \cdot \frac{1}{d} \cdot \frac{1}{d} = \frac{1}{d^3}$$

$$\therefore \text{Volume of solids} \quad (V_s) = \left( \frac{\pi}{6} d^3 \right) \times \frac{1}{d^3} = \frac{\pi}{6}$$

$$\therefore \text{Volume of voids} \quad (V_v) = 1 - \frac{\pi}{6}$$

$$\therefore \text{Voids ratio, } e = \frac{1 - \frac{\pi}{6}}{\frac{\pi}{6}} = 0.9099$$

$$\text{Porosity, } n = \frac{e}{1 + e} = \frac{0.9099}{1 + 0.9099} = 0.4764 = \mathbf{47.64\%}.$$

**Example 2.16.** Determine the minimum possible voids ratio of a uniformly graded sand of perfectly spherical grains.

**Solution :** Consider a *unit cube* of soil having spherical particles of diameter  $d$ . The soil will have maximum possible voids in this packing wherein the angle  $\alpha = 90^\circ$  (Fig. 2.7 a). Now rearrange the spheres so that the sphere centres form a rhombohedral array with angle  $\alpha = 60^\circ$ . The soil will attain densest state of packing in this arrangement. By such an *rearrangement* the volume of solids will remain the same, while the volume of voids will change, thus changing the total volume  $V$ .

If  $d$  is the diameter of each sphere, each of volume  $\pi d^3/6$ , the number of solids in the original unit cube  $= \frac{1}{d} \cdot \frac{1}{d} \cdot \frac{1}{d} = \frac{1}{d^3}$

$$\therefore \text{Volume of solids, } V_s = \frac{\pi}{6} d^3 \times \frac{1}{d^3} = \frac{\pi}{6}$$

Now the changed volume of rhombohedron is given by Eq. 2.36

$$V = \sqrt{1 + 2 \cos \alpha} (1 - \cos \alpha)$$

Here, for densest state of packing,  $\alpha = 60^\circ$

$$\therefore V = \sqrt{1 + 2 \cos 60^\circ} (1 - \cos 60^\circ) = 0.7071$$

Thus, the original *unit volume* is changed to 0.7071 due to densest packing. Since the volume of solids remain the same, we have  $V_s = \frac{\pi}{6} = 0.5236$

$$\therefore \text{Volume of voids, } V_v = V - V_s = 0.7071 - 0.5236 = 0.1835$$

$$\therefore \text{Voids ratio} \quad e = \frac{V_v}{V_s} = \frac{0.1835}{0.5236} = 0.35$$

$$\text{Hence porosity} \quad n = \frac{e}{1 + e} = \frac{0.35}{1 + 0.35} = 0.2595 = \mathbf{25.95\%}$$

**Example 2.17.** Determine the voids ratio and porosity of a uniformly graded sand of perfectly spherical grains arranged to form a rhombohedral array with angle of orientation of rhombohedron equal to  $75^\circ$ .

**Solution :** Refer Fig. 2.7 (b), where  $\alpha$  is equal to  $75^\circ$  (instead of  $60^\circ$ ).

The volume of original unit cube will change to new volume  $V$  due to this arrangement, without changing the initial volume of solids,  $V_s (= \pi/6)$ .

New volume is given by Eq. 2.36.

$$V = \sqrt{1 + 2 \cos \alpha (1 - \cos \alpha)} = \sqrt{1 + 2 \cos 75^\circ (1 - \cos 75^\circ)} = 0.9131$$

$$V_s = \pi/6 = 0.5236.$$

Hence  $V_v = V - V_s = 0.9131 - 0.5236 = 0.3895$

Hence voids ratio  $e = \frac{V_v}{V_s} = \frac{0.3895}{0.5236} = 0.7439$

and porosity  $n = \frac{e}{1 + e} = \frac{0.7439}{1 + 0.7439} = 0.4266 = 42.66\%$

## 2.8. EXAMPLES FROM COMPETITIVE EXAMINATIONS

**Example 2.18.** The soil fill for a road embankment is to be compacted in place to a void ratio of 0.7. If the void ratio of the borrow pit soil is 1.2, how many cubic metres of compacted fill can be placed in the embankment per 1000 cubic metres of borrow materials? If no water is either added or lost during the placement of the fill, what would be the percentage change in the degree of saturation of the soil? The compacted fill is not fully saturated. (Civil Services Exam. 1982)

**Solution:** Use suffix 1 for borrow pit soil and 2 for embankment fill soil.

$$e = \frac{V_v}{V_s}; \quad 1 + e = 1 + \frac{V_v}{V_s} = \frac{V_s + V_v}{V_s} = \frac{V}{V_s}. \quad \text{Hence } V = V_s (1 + e)$$

Since  $V_s$  remains constant,  $\frac{V_2}{V_1} = \frac{1 + e_2}{1 + e_1} \quad \therefore \quad V_2 = V_1 \left( \frac{1 + e_2}{1 + e_1} \right) = \frac{1000 (1 + 0.7)}{(1 + 1.2)} = 772.73 \text{ m}^3$

Now  $w = M_w/M_d$ . Since neither  $M_d$  changes nor  $M_w$  changes, the water content also remains the same, i.e.,  $w_1 = w_2$ . Only, volume changes from  $V_1$  to  $V_2$  due to expulsion of air voids.

$$\therefore \quad S_1 = \frac{wG}{e_1} \quad \text{and} \quad S_2 = \frac{wG}{e_2} \quad \therefore \quad \frac{S_1}{S_2} = \frac{e_2}{e_1}$$

$$\therefore \quad \% \text{ change is } S = \left( \frac{S_2 - S_1}{S_1} \right) 100 = \left( \frac{S_2}{S_1} - 1 \right) 100 = \left( \frac{e_1}{e_2} - 1 \right) 100 = \left( \frac{1.2}{0.7} - 1 \right) 100 = 71.43\%$$

**Example 2.19.** Test on a fill reveal that one cubic metre of soil in the fill weighs 1624 kg and after being dried, 1.40 tonnes. If the specific gravity of solids is 2.65, determine the water content, void ratio, porosity and degree of saturation of the soil mass in moist state. (Civil Services Exam. 1983)

**Solution**  $M_w = 1624 - 1400 = 224 \text{ kg}$  ;  $V_w = 224/1000 = 0.224 \text{ m}^3$

$$M_s = M_d = 1400 \text{ kg} ; \quad V_s = \frac{M_d}{G \rho_w} = \frac{1400 \times 1000}{2.65 \times 1} = 528302 \text{ cm}^3 = 0.5283 \text{ m}^3$$

$$\therefore \quad V_a = 1 - V_w - V_s = 1 - 0.224 - 0.5283 = 0.2477 \text{ m}^3 ; \quad V_v = V_a + V_w = 0.2477 + 0.2240 = 0.4717 \text{ m}^3$$

(i) Moisture content,  $w = M_w/M_d = 224/1400 = 0.16 = 16\%$

(ii) Voids ratio,  $e = V_v/V_s = 0.4717/0.5283 = 0.893$

(iii) Degree of saturation  $S = \frac{G \cdot w}{e} = \frac{2.65 \times 0.16}{0.893} = 0.4749 = 47.49\%$



$$(iv) \text{ Porosity } n = \frac{e}{1+e} = \frac{0.893}{1+0.893} = 0.472$$

**Example 2.20.** A soil sample is partially saturated. Its natural moisture content was found to be 22% and bulk density  $2 \text{ g/cm}^3$ . If the specific gravity of solid particles is 2.65 and the density of water be taken as  $1 \text{ g/cm}^3$ , find out the degree of saturation and the void ratio. (Civil Services Exam, 1984)

**Solution :** Given  $w = 0.22$  ;  $\rho = 2 \text{ g/cm}^3$  ;  $G = 2.65$  ;  $\rho_w = 1 \text{ g/cm}^3$

$$\text{From Eq. 2.28 (a),} \quad \rho = \frac{(G + e S_r) \rho_w}{1 + e} = \frac{G (1 + w) \rho_w}{1 + e}$$

$$\text{From which} \quad e = \frac{G \rho_w (1 + w)}{\rho} - 1 = \frac{2.65 \times 1 (1 + 0.22)}{2} - 1 = 0.6165$$

$$\text{Also from Eq. 2.20,} \quad S = \frac{wG}{e} = \frac{0.22 \times 2.65}{0.6165} = 0.9457 = 94.57\%$$

**Example 2.21.** A natural soil sample has a bulk density of  $2 \text{ g/cm}^3$  with 6% water content. Calculate the amount of water required to be added to one cubic metre of soil to raise the water content to 15% while the voids ratio remains constant. What is then the degree of saturation ?  $G = 2.67$ .

(Civil Services Exam, 1986)

**Solution :** Given  $\rho = 2 \text{ g/cm}^3$  ,  $w = 0.06$  ;  $G = 2.67$

$$\rho_d = \frac{\rho}{1 + w} = \frac{2}{1 + 0.06} = 1.887 \text{ g/cm}^3$$

**First stage :** When  $w = 6\% = 0.06 = \frac{M_w}{M_d}$ . Consider a unit volume  $V = 1 \text{ m}^3 = 10^6 \text{ cm}^3$

$$\text{Now,} \quad M_d = \rho_d \cdot V = 1.887 \times 10^6 \text{ g.} \quad \therefore \quad M_w = w M_d = 0.06 \times 1.887 \times 10^6 = 113220 \text{ g}$$

$$V_w = \frac{M_w}{\rho_w} = \frac{113220}{1} = 11.3220 \times 10^4 \text{ cm}^3$$

**Second stage:** When  $w = 15\% = 0.15$   $\therefore M_w = w M_d = 0.15 \times 1.887 \times 10^6 \text{ g} = 28.305 \times 10^4 \text{ g}$

$$\therefore V_w = \frac{M_w}{\rho_w} = \frac{28.305 \times 10^4}{1} = 28.305 \times 10^4 \text{ cm}^3.$$

$\therefore$  Additional volume of water required to raise  $w$  from 6% to 15%.

$$= 28.305 \times 10^4 - 11.3220 \times 10^4 = 16.983 \times 10^4 \text{ cm}^3 = 169.83 \text{ litres}$$

$$\text{Now voids ratio } e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.67 \times 1}{1.887} - 1 = 0.415$$

After adding water,  $e$  remains the same. Hence  $S = \frac{wG}{e} = \frac{0.15 \times 2.67}{0.415} = 0.9652 = 96.52\%$

**Example 2.22.** A mass of soil coated with thin layer of paraffin wax weighs 690.6 g and the soil alone weighs 683 g. When the sample is immersed in water, it displaces 350 ml of water. The specific gravity of the soil is 2.73 and that of wax is 0.89. Find out void ratio and degree of saturation if it has got water content of 17%. Unit weight of water is  $1000 \text{ kg/cu.m.}$  (Civil Services Exam, 1990)

**Solution :** Given :  $M_1 = 683 \text{ g}$  ;  $M_2 = 690.6 \text{ g}$  ;  $G_p = 0.89$  ;  $G = 2.73$

$$\therefore M_p = M_2 - M_1 = 690.6 - 683 = 7.6 \text{ g} \quad \therefore V_p = M_p / G_p = 7.6 / 0.89 = 8.539 \text{ cm}^3$$

$$V_w = 350 \text{ cm}^3 \text{ (given) ; } V = V_w - V_p = 350 - 8.539 = 341.461 \text{ cm}^3$$

$$\therefore \rho = M_1 / V = 683 / 341.461 = 2 \text{ g/cm}^3 \quad \text{or } w = 0.17.$$

$$\text{Hence } \rho_d = \rho / (1 + w) = 2 / (1 + 0.17) = 1.709 \text{ g/cm}^3$$

Now  $\rho_w = 1000 \text{ kg/m}^3 = 1 \text{ g/cm}^3$ ,  $\therefore e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.73 \times 1}{1.709} - 1 = \mathbf{0.597}$

and  $S = \frac{w G}{e} = \frac{0.17 \times 2.73}{0.597} = 0.7773 = \mathbf{77.73\%}$

**Example 2.23.** A compacted cylindrical specimen 50 mm diameter and 100 mm long is to be prepared from dry soil. If the specimen is required to have a water content of 15% and the percentage of air voids is 20, calculate the weight of soil and water required in the preparation of the soil whose specific gravity is 2.69. (Civil Services Exam, 1991)

**Solution :** Given  $w = 15\%$  ;  $a_v = 20\%$  ;  $G = 2.69$

$$V = \frac{\pi}{4} (50)^2 \times 10 = 196.35 \text{ cm}^3 ; \quad M_w = 0.15 M_d ; V_a = 0.20 V$$

Now  $V_w = \frac{M_w}{\rho_w} = \frac{0.15 M_d}{\rho_w} = \frac{0.15 (V_s \cdot G \rho_w)}{\rho_w} = 0.15 \times 2.69 V_s = 0.4035 V_s$

Now  $V = V_a + V_w + V_s = 0.20 V + 0.4035 V_s + V_s$

or  $0.8 V = 1.4035 V_s$ , from which  $V_s = 0.57 V = 0.57 \times 196.35 = 111.92 \text{ cm}^3$

$\therefore M_d = V_s G \rho_w = 111.92 \times 2.69 \times 1 = \mathbf{301.1 \text{ g}}$

Also,  $M_w = 0.15 M_d = 0.15 \times 301.1 = \mathbf{45.2 \text{ g}}$

**Example 2.24.** A clayey soil has saturated moisture content of 15.8%. The specific gravity is 2.72. Its saturation percentage is 70.8%. The soil is allowed to absorb water. After some time, the saturation increased to 90.8%. Find the water content of the soil in the latter case. (Civil Services Exam, 1992)

**Solution :** Given :  $w = 15.8\%$  ;  $G = 2.72$  ;  $S = 70.8\%$

Initially,  $e = \frac{wG}{S} = \frac{0.158 \times 2.72}{0.708} = 0.607$

When the soil is allowed to absorb water, the air voids are replaced partially (or fully) by water and hence the voids ratio remains constant.

Now, after absorption,  $e = \frac{wG}{S} = 0.607$  ;  $\therefore w = \frac{0.607 \times S}{G} = \frac{0.607 \times 0.908}{2.72} = 0.2026 = \mathbf{20.26\%}$

**Example 2.25.** A core cutter 12.6 cm in height and 10.2 cm in diameter weighs 1071 gm when empty. It is used to determine the in-situ unit weight of an embankment. The weight of core cutter full of soil is 2970 gm. If the water content is 6%, what are the in-situ dry weight and porosity? (ii) if the embankment gets fully saturated due to heavy rains what will be the increase in water content and bulk unit weight, if no volume change occurs? The specific gravity of soil solids is 2.69.

(Civil Services Exam, 1997)

**Solution :**  $V = \frac{\pi}{4} D^2 H = \frac{\pi}{4} (10.2)^2 \times 12.6 = 1029.58 \text{ cm}^3$

Mass of soil sample,  $M = 2970 - 1071 = 1899 \text{ cm}^3$

$\therefore \rho = M/V = 1899/1029.58 = 1.844 \text{ g/cm}^3$  ;  $\gamma = 1.844 \times 9.81 = 18.094 \text{ kN/m}^3$

$\rho_d = \frac{\rho}{1+w} = \frac{1.844}{1+0.06} = 1.74 \text{ g/cm}^3$   $\therefore \gamma_d = 1.74 \times 9.81 \approx \mathbf{17.07 \text{ kN/m}^3}$

$e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.69 \times 9.81}{17.07} - 1 = \mathbf{0.546}$

(ii) When the embankment gets saturated,  $e$  remain the same, since water enter by expelling air from the voids.

Now  $e = w_{sat} \cdot G$   $\therefore w_{sat} = \frac{e}{G} = \frac{0.546}{2.69} = 0.203 = \mathbf{20.3\%}$

$\therefore$  Increase in water content  $= 0.203 - 0.06 = 0.143 = \mathbf{14.3\%}$

Also,  $\gamma_{sat} = \gamma_d (1 + w_{sat}) = 17.07 (1 + 0.203) = \mathbf{20.535 \text{ kN/m}^3}$



$\therefore$  Increase in bulk unit weight  $= 20.535 - 18.094 = 2.441 \text{ kN/m}^3$

**Example 2.26.** A highly sensitive volcanic clay was investigated in the laboratory and found to have the following properties :

(i)  $\gamma_{wet} = 12.50 \text{ kN/m}^3$

(ii)  $G = 2.75$

(iii)  $e = 9.0$

(iv)  $w = 311\%$

In rechecking the above values, one was found to be inconsistent with the rest. Find the inconsistent value and report it correctly. (Civil services Exam. 2001)

**Solution :** The solution will be done by trial.

**Assumption (i) :** Let  $\gamma_{wet}$  be the inconsistent value, and the values of  $G$ ,  $e$  and  $w$  be correct.

Then 
$$S = \frac{wG}{e} = \frac{3.11 \times 2.75}{9.0} = 0.95 < 1 \text{ (Hence OK)}$$

From Eq. 2.28, 
$$\gamma = \frac{(G + eS) \gamma_w}{1 + e} = \frac{(2.75 + 9.0 \times 0.95) 9.81}{1 + 9.0} = 11.09 \text{ kN/m}^3$$

Since this computed value of  $\gamma$  does not match with the given value of  $12.56 \text{ kN/m}^3$ , the given value is inconsistent. In order to show that  $\gamma_{wet}$  is the only inconsistent value, let us check back by assuming any of the remaining value to be inconsistent.

**Assumption (ii) :** Let value of  $G$  be inconsistent, and let  $\gamma_{wet}$ ,  $e$  and  $w$  be correct.

$$\therefore \gamma_d = \frac{\gamma_{wet}}{1 + w} = \frac{12.56}{1 + 3.11} = 3.06 \text{ kN/m}^3$$

Hence 
$$G = \frac{\gamma_d (1 + e)}{\gamma_w} = \frac{3.06 (1 + 9.0)}{9.81} = 3.115 \text{ and } S = \frac{wG}{e} = \frac{3.11 \times 3.115}{9.0} = 1.076 > 1$$

Since  $S$  is more than 1, the above assumption is not feasible, i.e.  $G$  is not inconsistent.

**Assumption (iii) :** Let the value of  $e$  be inconsistent and let  $\gamma_{wet}$ ,  $G$  and  $w$  be correct.

Then, 
$$\gamma_d = \frac{\gamma_{wet}}{1 + w} = \frac{12.53}{1 + 3.11} = 3.06 \text{ kN/m}^3$$

$$\therefore e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.75 \times 9.81}{3.06} - 1 = 7.828 \text{ and } S = \frac{wG}{e} = \frac{3.11 \times 2.75}{7.828} = 1.093 > 1.$$

Since  $S$  is more than 1, the above assumption is not feasible, i.e. the value of  $e (= 9.0)$  is not inconsistent.

**Assumptions (iv) :** Lastly, let  $w$  be incorrect and  $\gamma_{wet}$ ,  $G$  and  $e$  be correct.

Now 
$$\gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.75 \times 9.81}{1 + 9.0} = 2.698 \text{ kN/m}^3$$

$$\therefore w = \frac{\gamma_{wet}}{\gamma_d} - 1 = \frac{12.56}{2.698} - 1 = 3.655 \text{ and } S = \frac{wG}{e} = \frac{3.655 \times 2.75}{9.0} = 1.117 > 1$$

Since  $S$  is more than 1, the above assumption is incorrect, i.e. the value of  $w$  is not inconsistent.

**Final result :** Hence only assumption (i) is correct, i.e. the value of  $\gamma_{wet}$  is inconsistent. The correct value of  $\gamma_{wet} = 11.09 \text{ kN/m}^3$ .

**Example 2.27.** The unit weight of a sand backfill was determined by field measurements to be  $1746 \text{ kg per cu. m}$ . The water content at the time of test was  $8.6 \text{ percent}$  and the unit weight of the solid constituents was  $2.6 \text{ gm per cu cm}$ . In the laboratory, the void ratios in the loosest and densest states were found to be  $0.642$  and  $0.462$  respectively. What was the relative density of the fill ? Write the importance of this term. (U.P.S.C. Engg. Services Exam. 1994)

**Solution :** Give  $\rho = 1746 \text{ kg/m}^3 = 1.746 \text{ g/cm}^3$ ;  $w = 0.086$ ;  $\gamma_s = 2.6 \text{ g/cm}^3$ .

$$\therefore G = \frac{\gamma_s}{\gamma_w} = \frac{2.6}{1} = 2.6 ; \quad \text{Now} \quad \rho = \frac{G \rho_w (1 + w)}{1 + e}$$

$$\text{From which} \quad e = \frac{G \rho_w (1 + w)}{\rho} - 1 = \frac{2.6 \times 1 (1 + 0.086)}{1.746} - 1 = 0.617$$

$$\text{Also, given} \quad e_{\max} = 0.642 \quad \text{and} \quad e_{\min} = 0.462$$

$$\therefore I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} = \frac{0.642 - 0.617}{0.642 - 0.462} = 0.1379 = 13.79\%$$

Such a low value of  $I_D$  indicates that the soil is very loose.

**Example 2.28.** : Soil is to be excavated from a borrow pit which has a density of 1.75 gm/c.c. and water content of 12%. The specific gravity of soil particles is 2.7. The soil is compacted so that water content is 18% and dry density is 1.65 gm/c.c. For 1000 cum of soil in fill, estimate (i) the quantity of soil to be excavated from the pit in cum; and (ii) the amount of water to be added. Also determine the void ratios of the soil in borrow pit and fill. (Engg. Service Exam., 1995)

**Solution** : Given  $\gamma = 1.75 \text{ g/cm}^3$ ;  $w = 12\%$ ;  $G = 2.7$ . After compaction,  $\gamma_d = 1.65 \text{ g/cm}^3$ ;  $w = 18\%$

Let us use suffix 1 for borrow pit and 2 for the fill.

$$\text{For the Borrow pit} \quad \rho = \frac{G \rho_w (1 + w)}{1 + e} \quad \text{or} \quad e_1 = \frac{G \rho_w (1 + w)}{\rho} - 1 = \frac{2.7 \times (1 + 0.12)}{1.75} - 1 = 0.728$$

$$\text{For the fill} \quad \rho_d = \frac{G \rho_w}{1 + e} \quad \text{or} \quad e_2 = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.7 \times 1}{1.65} - 1 = 0.6364$$

Since the volume of solids remains constant,

$$V_s = \frac{V_1}{1 + e_1} = \frac{V_2}{1 + e_2} = \frac{1000 \text{ m}^3}{1 + 0.6364} \approx 611.1 \text{ m}^3 \quad \therefore V_1 = 611.1 (1 + 0.728) \approx 1056 \text{ m}^3$$

Again, in general,  $w = M_w/M_d$

$$\text{But} \quad M_d = M_s = \text{mass of solids} = V_s G \rho_w = 611.1 \times 2.7 \times 1 = 1649.97 \text{ t}$$

$$\text{Now,} \quad M_{w1} = M_d \cdot w_1 = 1649.97 \times 0.12 = 197.996 \text{ m}^3 ; \quad M_{w2} = M_d \cdot w_2 = 1649.97 \times 0.18 = 296.995 \text{ m}^3$$

$$\therefore \text{Water to be added} = M_{w2} - M_{w1} = 296.995 - 197.996 \approx 99 \text{ m}^3$$

**Example 2.29.** A sampler with a volume of 45 cm<sup>3</sup> is filled with a soil sample. When the soil is poured into a graduated cylinder, it displaces 25 cm<sup>3</sup> of water. What is the porosity and void ratio of the soil? (Engg. Services Exam., 1998)

**Solution:** Given  $V = 45 \text{ cm}^3$ ; Volume of solids,  $V_s =$  Volume of water displaced  $= 25 \text{ cm}^3$

$$\therefore V_v = V - V_s = 45 - 25 = 20 \text{ cm}^3 \quad \therefore e = V_v/V_s = 20/25 = 0.8$$

$$n = V_v/V = 20/45 = 0.444$$

**Example 2.30.** The void ratio and specific gravity of a sample of clay are 0.73 and 2.7 respectively. If the voids are 92% saturated, find the bulk density, dry density and the water content. What would be the water content for complete saturation, the void ratio remaining the same?

(Engg. Services Exams, 1999)

**Solution** : Given :  $e = 0.73$ ;  $G = 2.7$ ;  $S = 92\%$

$$w = \frac{eS}{G} = \frac{0.73 \times 0.92}{2.7} = 0.2487 = 24.87\% ; \quad \gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.7 \times 9.81}{1 + 0.73} = 15.31 \text{ kN/m}^3$$

$$\gamma = \gamma_d (1 + w) = 15.31 (1 + 0.2487) = 19.118 \text{ kN/m}^3 ; \quad w_{\text{sat}} = \frac{e}{G} = \frac{0.73}{2.7} = 0.2703 = 27.03\%$$

**Example 2.31.** A sample of sand above water table was found to have a natural moisture content of 15% and a unit weight of 18.84 kN/m<sup>3</sup>. Laboratory tests on a dried sample indicated values of 0.5



and 0.85 for minimum and maximum void ratios, respectively for densest and loosest states. Calculate the degree of saturation and the relative density. Assume  $G = 2.65$ . (Engg. Services Exam, 2000)

**Solution :** Given  $w = 15\%$ ,  $\gamma = 18.84 \text{ kN/m}^3$ ;  $e_{\min} = 0.5$ ;  $e_{\max} = 0.85$

$$\text{From Eqs. 2.24 and 2.31, } e = \frac{G \gamma_w (1 + w)}{\gamma} - 1 = \frac{2.65 \times 9.81 (1 + 0.15)}{18.84} - 1 = 0.587$$

$$\text{From Eq. 2.20, } S = \frac{wG}{e} = \frac{0.15 \times 2.65}{0.587} = 0.6774 = 67.74\%$$

$$\text{Also, } I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} = \frac{0.85 - 0.587}{0.85 - 0.5} = 0.7514 = 75.14\%$$

**Example 2.32.** Cohesive soil yields a maximum dry density of  $1.8 \text{ g/cc}$  at an OMC of  $16\%$  during a standard Proctor test. If the value of  $G$  is  $2.65$ , what is the degree of saturation? What is the maximum dry density it can be further compacted to? (Gate Exam, 1992)

**Solution :** Given  $\rho_d = 1.8 \text{ g/cm}^3$ ;  $w = 16\%$ ;  $G = 2.65$

$$e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.65 \times 1}{1.8} - 1 = 0.472 \quad \therefore S = \frac{wG}{e} = \frac{0.16 \times 2.65}{0.472} = 0.8979 = 89.79\%$$

Theoretically  $\gamma_{d, \max}$  is obtained at zero air voids, i.e. when  $S = 1$ .

$$\therefore \rho_{d, \max} = \frac{G \rho_w}{1 + wG} = \frac{2.65 \times 1}{1 + 0.16 \times 2.65} = 1.861 \text{ g/cm}^3$$

**Example 2.33.** The total unit weight ( $\gamma_t$ ) of soil is  $18.8 \text{ kN/m}^3$  the specific gravity ( $G$ ) of the solid particles of soil is  $2.67$  and the water content ( $w$ ) of the soil is  $12\%$ . Calculate the dry unit weight ( $\gamma_d$ ), the void ratio ( $e$ ) and the degree of saturation. (Gate Exam, 1994)

**Solution :**  $\gamma_t (= \gamma) = 18.8 \text{ kN/m}^3$ ;  $G = 2.67$ ;  $w = 12\%$ ; Now  $\gamma_d = \frac{\gamma_t}{1 + w} = \frac{18.8}{1 + 0.12} = 16.786 \text{ kN/m}^3$

$$\therefore e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{16.786} - 1 = 0.56 \quad \text{and} \quad S = \frac{wG}{e} = \frac{0.12 \times 2.67}{0.56} = 0.5717 = 57.17\%$$

**Example 2.34.** The total unit weight of the glacial outwash soil is  $16 \text{ kN/m}^3$ . The specific gravity of soil particles of the soil is  $2.67$ . The water content of the soil is  $17\%$ . Calculate :

(a) Dry unit weight

(b) Porosity

(c) Void ratio

(d) Degree of saturation

(Gate Exam, 1998)

**Solution :**  $\gamma_t (= \gamma) = 16 \text{ kN/m}^3$ ;  $G = 2.67$ ;  $w = 17\%$

$$\gamma_d = \frac{\gamma}{1 + w} = \frac{16}{1 + 0.17} = 13.675 \text{ kN/m}^3;$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{13.675} - 1 = 0.9153$$

$$n = \frac{e}{1 + e} = \frac{0.9153}{1 + 0.9153} = 0.4779 = 47.79\% ;$$

$$S = \frac{wG}{e} = \frac{0.17 \times 2.67}{0.9153} = 0.4959 = 49.59\%$$

**Example 2.35.** Soil has been compacted in an embankment at a bulk density of  $2.15 \text{ Mg/m}^3$  and water content of  $12\%$ . The value of specific gravity of soil solids is  $2.65$ . The water table is well below the foundation level. Estimate the dry density, void ratio, degree of saturation and air content of compacted soil. (Gate Exam, 2002)

**Solution :** Given  $\rho = 2.15 \text{ Mg/m}^3 = 2.15 \text{ g/cm}^3$ ;  $w = 12\%$ ;  $G = 2.65$

$$\rho_d = \frac{\rho}{1 + w} = \frac{2.15}{1 + 0.12} = 1.92 \text{ g/cm}^3 ;$$

$$e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.65 \times 1}{1.92} - 1 = 0.38$$

$$S = \frac{wG}{e} = \frac{0.12 \times 2.65}{0.38} = 0.8358 = 83.58\% ; \quad a_c = 1 - S = 1 - 0.8358 = 0.1642 = 16.42\%$$

## PROBLEMS

1. The natural bulk unit weight of a sandy stratum is  $18.54 \text{ kN/m}^3$  and it has a water content of 8%. For determining the density index, dried sand from the stratum was first filled loosely in a  $300 \text{ cm}^3$  mould and then vibrated to give a maximum density. The loose dry mass in the mould was 480 g and the dense dry mass at the maximum compaction was 570 g. If the specific gravity of solids is 2.66, find the density index of the sand in the stratum.

[Ans.  $I_D = 0.54$ ]

2. A saturated specimen of undisturbed inorganic clay has a volume of  $19.2 \text{ cm}^3$  and mass 32.5 g. After oven-drying at  $105^\circ \text{C}$  for 24 hours, the mass reduces to 20.9 g. For the soil in the natural state, find (i)  $w$ , (ii)  $G$ , (iii)  $e$ , (iv)  $\gamma_{sat}$  and (v)  $\gamma_d$ .

[Ans. (i) 55.4%, (ii) 2.76, (iii) 1.53, (iv)  $16.68 \text{ kN/m}^3$ , (v)  $10.69 \text{ kN/m}^3$ ]

3. In a Jodhpur-Mini-Compactor test, 612 g of wet soil occupies a volume of  $300 \text{ cm}^3$  at a moisture content of 12.6%. Determine (i)  $\gamma$ , (ii)  $\gamma_d$ , (iii)  $e$ , (iv)  $n$  and (v)  $S$  in the compacted soil if the specific gravity of soil solids is 2.68.

[Ans. (i)  $20.01 \text{ kN/m}^3$ , (ii)  $17.76 \text{ kN/m}^3$ , (iii) 0.48, (iv) 32.4% and (v) 70.3%]

4. A moist sample of soil has a mass of 633 g and a volume of  $300 \text{ cm}^3$  at a water content of 11%. Taking  $G = 2.68$ , determine  $e$ ,  $S$  and  $n_a$ . Also determine the water content at which the soil gets fully saturated without any increase in the volume. What will be the unit weight at saturation?

[Ans.  $e = 0.41$  ;  $S = 71.9\%$  ;  $n_a = 8.2\%$  ;  $w_{sat} = 15.3\%$  ;  $\gamma_{sat} = 21.48 \text{ kN/m}^3$ ]

5. A compacted sample of soil with a bulk unit weight of  $19.62 \text{ kN/m}^3$  has a water content of 15 per cent. What are its dry density, degree of saturation and air-content? Assume  $G = 2.65$ .

[Ans.  $\gamma_d = 17.07 \text{ kN/m}^3$ ,  $S = 76\%$  ;  $a_c = 24\%$ ]

6. A saturated sample of soil has a water content of 35%. Adopting  $G = 2.70$ , calculate  $\gamma_d$ ,  $\gamma_{sat}$  and  $\gamma'$ .

[Ans.  $\gamma_d = 13.64 \text{ kN/m}^3$ ,  $\gamma_{sat} = 18.44 \text{ kN/m}^3$ ,  $\gamma' = 8.63 \text{ kN/m}^3$ ]

7. A fully saturated clay sample has mass of 101.5 g and a volume of  $50 \text{ cm}^3$ . After oven drying, the clay has mass of 84.5 g. Assuming that the volume does not change during drying, determine (i)  $G$ , (ii)  $e$ , (iii)  $n$  and (iv)  $\gamma_d$ .

[Ans.  $G = 2.55$ ,  $e = 0.51$ ,  $n = 33.8\%$ ,  $\gamma_d = 16.58 \text{ kN/m}^3$ ]

8. By three phase soil system, show that the degree of saturation  $S$  (as ratio) in terms of mass unit weight  $\gamma$ , water content  $w$  (as ratio), specific gravity of soil grains  $G$ , and unit weight of water  $\gamma_w$ , is given by the expression

$$S = \frac{w}{\frac{\gamma_w}{\gamma} (1 + w) - \frac{1}{G}}$$

9. An imaginary soil mass is contained in a container measuring  $10 \text{ cm} \times 10 \text{ cm} \times 10 \text{ cm}$ . The soil consists of spherical grains of size 1 cm in diameter. Determine the maximum possible voids ratio, porosity and per cent solids.

[Ans. 0.91 ; 47.6% ; 52.4%]

## Determination of Index Properties

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### 3.1. GENERAL

In this chapter, we shall describe the methods of determining those properties of soils which are used in their identification and classification. These include the determination of : (i) water content, (ii) specific gravity, (iii) particle size distribution, (iv) consistency limits, (v) in-situ density and (vi) density index. These properties are known as *index properties*.

### 3.2. WATER CONTENT

The water content of a soil sample can be determined by the following methods:

- |                               |                              |
|-------------------------------|------------------------------|
| (i) Oven drying method;       | (ii) Sand bath method        |
| (iii) Alcohol method;         | (iv) Calcium carbide method. |
| (v) Pycnometer method.;       | (vi) Radiation method.       |
| (vii) Torsion balance method. |                              |

1. **Oven drying method.** This is the most accurate method of determining the water content, and is, therefore, used in the laboratory. A specimen of soil sample is kept in a clean container and put in a thermostatically controlled oven with interior of non-corroding material to maintain the temperature between 105°C to 110°C. For complete drying, sandy soils take about four hours and fat clays take about 14 to 16 hours. Usually the sample is kept for about 24 hours in the oven so that complete drying is assured. A temperature higher than 110 ° C may break the crystalline structure of clay particles resulting in the loss of chemically bound structural water. For highly organic soils, such as peat, a lower temperature of about 60 °C is preferable to prevent the oxidation of the organic matter. Certain soils contain gypsum which on heating loses its water of crystallisation. If it is suspected that gypsum is present in the soil, the sample is dried at not more than 80°C but for a longer time (IS : 2720 Part II-1969).

A clean non-corrodible container is taken and its mass is found with its lid, on a balance accurate to 0.01 g. A specimen of the moist soil is placed in the container and the lid is replaced. The mass of the container and the contents is determined. With the lid removed, the container is then placed in the oven for drying. After drying, the container is removed from the oven and allowed to cool in a desiccator. The lid is then



replaced, and the mass of container and the dry soil is found. The water content is calculated from the following expression :

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \quad (\text{percent}) \quad \dots(3.1)$$

where  $M_1$  = mass of container with lid

$M_2$  = mass of container with lid and wet soil

$M_3$  = mass of container with lid and dry soil

The observations are tabulated as illustrated in Table 3.7, Experiment 1.

**2. Sand bath method.** This is a field method of determining rough value of the water content, where the facility of an oven is not available. The container with the soil is placed on a sand bath. The sand bath is heated over a kerosene stove. The soil becomes dry within 1/2 to 1 hour. The water content is then determined from Eq. 3.1. However, higher temperature may break the crystalline structure of soils. This method should not be used for organic soils, or for soils having higher percentage of gypsum (B.S. 1377 : 1961).

**3. Alcohol method.** This is also a crude field method. The wet soil sample is kept in a evaporating dish and mixed with sufficient quantity of methylated spirit. The dish is then properly covered and the mixture is ignited. The mixture is kept stirred by a wire during ignition. Since there is no control over the temperature it should not be used for soils containing large percentage of organic matter or gypsum. The water content is determined from the expression:

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \quad \text{where } M_1 = \text{mass of empty dish}$$

$M_2$  = mass of dish + wet soil ;  $M_3$  = mass of dish + dry soil.

**4. Calcium carbide method.** In this method, 6 g of wet soil sample is placed in an air-tight container (called moisture tester) and is mixed with sufficient quantity of fresh calcium carbide powder. The mixture is shaken vigorously. The acetylene gas, produced by the reaction of the moisture of the soil and the calcium carbide, exerts pressure on a sensitive diaphragm placed at the end of the container. The dial gauge located at the diaphragm reads the water content directly. However, the calibration of the dial gauge is such that it gives the water content ( $w'$ ) based on the wet weight of the sample. Knowing the water content  $w'$  based on wet weight, the water content ( $w$ ) based on dry weight can be found from Eq. 2.1 (d) :

$$w = \frac{w'}{1 - w'}$$

The method is very quick — the result can be obtained in 5 to 10 minutes. The field kit contains the moisture tester, a small single-pan weighing balance, a bottle containing calcium carbide and a brush. A big container permits the use of 26 g sample (Blysten : 1961). In order that balls, having wet soil inside and dry soil outside, may not form during the reaction with calcium carbide, the soil may be mixed with perfectly dry sand.

In the larger container, two 30 mm diameter balls are placed for proper pulverisation of clay soils. This method is specially suited to a circumstance where water content is to be quickly determined for the purpose of proper field control, such as in the compaction of an embankment.

**5. Pycnometer method.** This is also a quick method of determining the water content of those soils whose specific gravity  $G$  is accurately known. Pycnometer (Fig. 3.1 a) is a large size density bottle of about 900 ml capacity. A conical brass cap, having a 6 mm diameter hole at its top is screwed to the open end of the pycnometer. A rubber washer is placed between conical cap and the rim of the bottle so that there is no leakage of water.

**Test procedure :**

1. Take a clean, dry pycnometer, and find its mass with its cap and washer ( $M_1$ ).
2. Put about 200 g to 400 g of wet soil sample in the pycnometer and find its mass with its cap and washer ( $M_2$ ).
3. Fill the pycnometer to half its height and mix it thoroughly with the glass rod. Add more water, and stir it. Replace the screw top and fill the pycnometer flush with the hole in the conical cap. Dry the pycnometer from outside, and find its mass ( $M_3$ ).
4. Empty the pycnometer, clean it thoroughly, and fill it with clean water to the hole of the conical cap, and find its mass ( $M_4$ ).

The water content is then calculated from the following expression :

$$w = \left[ \left( \frac{M_2 - M_1}{M_3 - M_4} \right) \left( \frac{G - 1}{G} \right) - 1 \right] \times 100 \quad \dots(3.2)$$

The above expression can be derived with reference to Fig. 3.2.

In Fig. 3.1 a (iii), if  $M_d$  is the mass of soil particles, the volume of solid particles will be equal to  $M_d/G$ . Thus, if the solids from (iii) are replaced with water of mass  $M_d/G$ , we get the mass  $M_4$  indicated in (iv). Thus,

$$M_4 = M_3 - M_d + \frac{M_d}{G} \quad \text{or} \quad M_d \left( \frac{G - 1}{G} \right) = M_3 - M_4$$

$$\text{From which,} \quad M_d = (M_3 - M_4) \frac{G}{G - 1} \quad \dots(i)$$

Now mass of water  $M_w$  in the wet soil sample =  $(M_2 - M_1) - M_d$

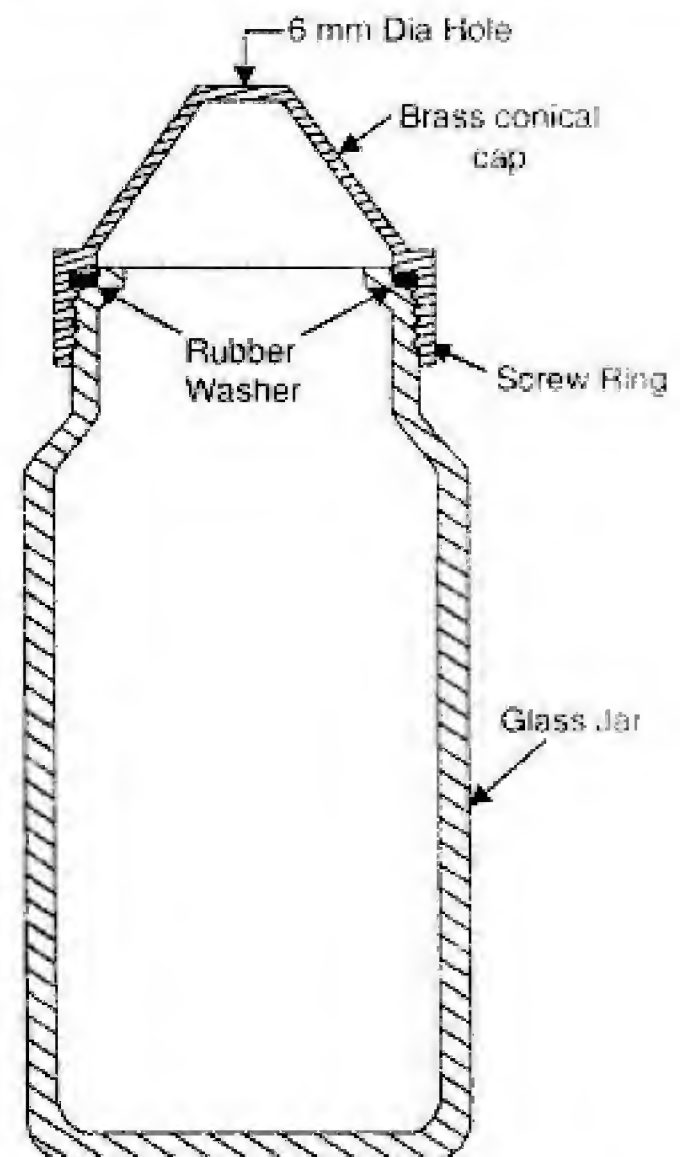


FIG. 3.1 (a). PYCNOMETER.

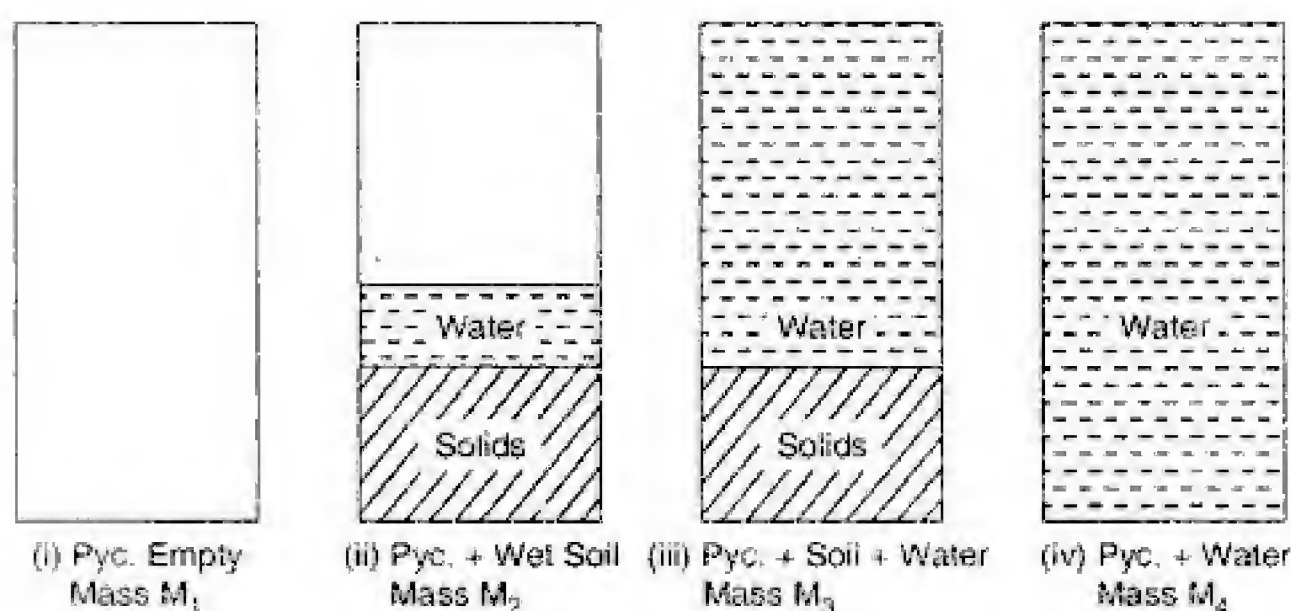


FIG. 3.1 (b) WATER CONTENT DETERMINATION.

$$\begin{aligned}
 \therefore w &= \frac{M_w}{M_d} \times 100 = \frac{M_2 - M_1 - M_d}{M_d} \times 100 \\
 &= \left( \frac{M_2 - M_1}{M_d} - 1 \right) 100 = \left[ \left( \frac{M_2 - M_1}{M_3 - M_4} \right) \left( \frac{G - 1}{G} \right) - 1 \right] \times 100 \quad \dots(3.2 \ a)
 \end{aligned}$$

This method is suitable for coarse grained soils only.

**6. Radiation method :** This method is extremely useful for the determination of water content of soil deposit in the in-situ condition. It uses two steel casings — casing *A* and casing *B* which are placed in two bore holes at some distance apart, in the soil deposit the field moisture content of which is to be determined. A device containing some radio-active isotope material (such as cobalt - 60) is placed in a capsule which in turn is lowered into casing *A*.

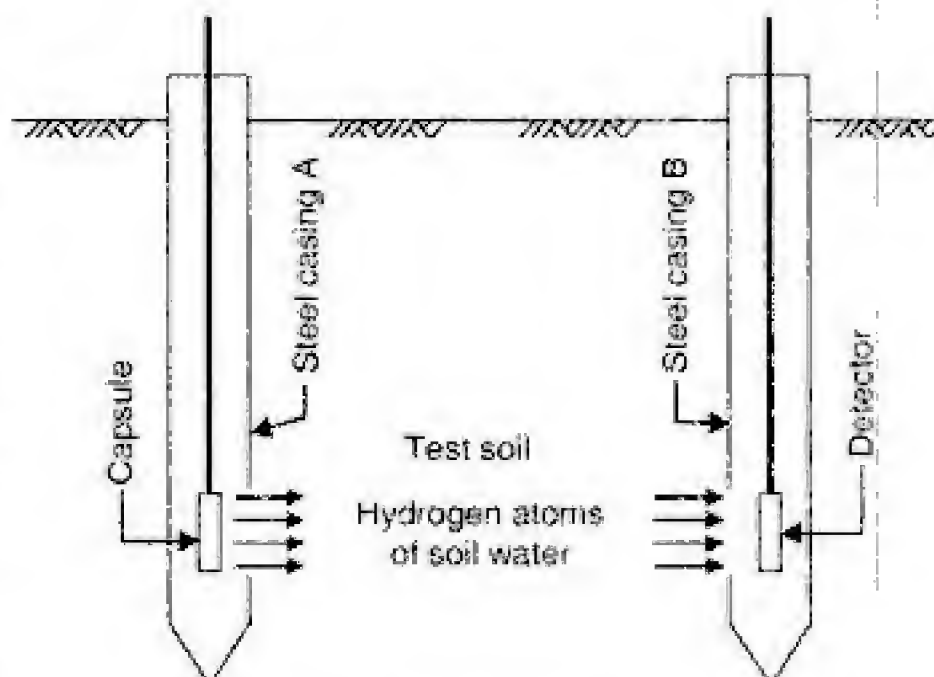


FIG. 3.2 RADIATION METHOD

Similarly, a detector unit is lowered in steel casing *B*. Small openings are made in both casing *A* and *B*, facing each other. When the radio-active device is activated, it emits neutrons. When these neutrons strike with the hydrogen atoms of water in the sub-soil, they loose energy. The loss of energy is evidently equal to water content in the soil. The detector device is calibrated to given directly the water content of the subsoil, at that level of emission. However, proper shielding precautions should be taken to avoid radiation problems.

**7. Torsion Balance method** (IS : 2720, Part II-1973) The equipment has two main parts : (i) *infra-red* lamp and (ii) torsion balance. The *infra-red* radiation is provided by 250 watt lamp built in the balance for use with alternating current 220-230 V, 50 cycle,



single phase mains supply. The weighing mechanism, a torsion balance, has a built-in magnetic damper to reduce pan vibrations during quick drying. The balance scale is divided in terms of water percentages from 1 to 100 water content in 0.2 percent division. The moisture meter is generally calibrated to use 25 g of soil, and hence the maximum size of particle present in the specimen should be less than 2 mm.

The test specimen is kept in a suitable container so that the water content to be determined is not affected by ambient conditions. Torque is applied to one end of the torsion wire by means of a calibrated drum to balance the loss of weight of the sample as it dries out under infrared lamp. To determine the percent reduction of mass at any instant, rotate the drum scale by turning the drum drive knob until the point returns to the index. The percent is read directly from the scale. However, this percent ( $w'$ ) is the percent of water based upon the initial mass (i.e wet mass) of the sample. The water content ( $w$ ) based on the dry mass is computed from Eq. 2.1 (d).

$$w = \frac{w'}{1 - w'}$$

Provision is made to adjust the input voltage to the infra-red lamp to control the heat for drying the specimen. A suitable thermometer graduated from 40° to 150° C is provided for ascertaining the temperature of drying in the pan housing. Normally, the temperature is kept between 110° ± 5°C. The time required for the test depends upon the type of soil and the quantity of water present; the normal time varies between 15 to 30 minutes. Drying and weighing occur simultaneously, and hence the method is specially suitable for those soils which re-absorb moisture quickly on drying. The criterion for taking the *final reading* is that the pointer should remain steady on index mark which shows that the sample has dried to constant mass.

### 3.3. SPECIFIC GRAVITY

The specific gravity of soil solids is determined by : (i) a 50 ml density bottle, or (ii) a 500 ml flask, or (iii) a pycnometer. The density bottle method is the most accurate, and is suitable for all types of soils. The flask or pycnometer is used only for coarse grained soils. The density bottle method is the standard method used in the laboratory.

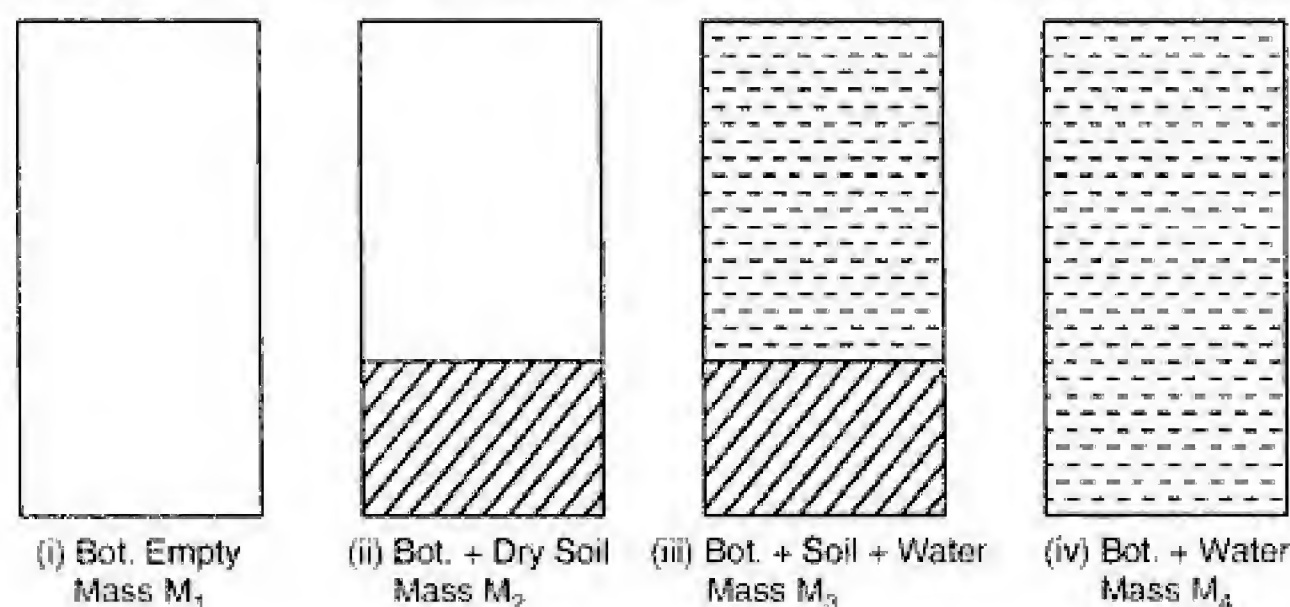


FIG. 3.3 SPECIFIC GRAVITY COMPUTATION.

However, in all the three methods, the sequence of observations is the same. The mass  $M_1$  of the empty, dry, bottle (or flask or pycnometer) is first taken. A sample of oven-dried soil, cooled in a desiccator, is put in the bottle, and the mass  $M_2$  is taken. The bottle is then filled with distilled water (or kerosene) gradually, removing the entrapped air either by applying vacuum or by shaking the bottle. The mass  $M_3$  of the bottle, soil and water (full up to the top) is taken. Finally, the bottle is emptied completely and thoroughly washed, and clean water (or kerosene) is filled to the top, and the mass  $M_4$  is taken. Based on these four observations, the specific gravity can be computed as follows (Fig. 3.3).

From Fig. 3.3 (i) and (ii), dry mass  $M_d$  of the soil is  $M_d = M_2 - M_1$  ... (a)

Mass of water in (iii) =  $M_3 - M_2$  ; Mass of water in (iv) =  $M_4 - M_1$

Hence, mass of water having the same volume as that of soil solids is  

$$= (M_4 - M_1) - (M_3 - M_2)$$

Now 
$$G = \frac{\text{Dry mass of soil}}{\text{Mass of water of equal volume}} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

or 
$$G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)} = \frac{M_d}{M_d - (M_3 - M_4)} \quad \dots (3.3)$$

**Alternative Proof.** If the soil solids are removed from  $M_3$  [Fig. 3.3 (iii)] and replaced by water of equal volume,  $M_4$  is obtained.

Volume of solids in (iii) =  $\frac{M_d}{G}$

$\therefore M_4 = M_3 - M_d + \frac{M_d}{G}$

or 
$$G = \frac{M_d}{M_d - (M_3 - M_4)} = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$

When a 500 ml flask is used, mass is taken to an accuracy of 0.1 g. In case of pycnometer, mass measurements are taken to 1 g accuracy. In both the cases, the entrapped air is expelled by stirring and distilled water is used. However, in the density bottle method, mass measurements are taken to an accuracy of 0.001 to 0.005 g and kerosene is used since it is a better wetting agent. In this case, specific gravity of soil solids is given by

$$G = \frac{M_d \cdot G_k}{M_d - (M_3 - M_4)} \quad \dots (3.3a)$$

where  $G_k$  is the specific gravity of kerosene at the test temperature  $T_t^\circ\text{C}$ .

When specific gravity of a cohesive soil is determined, the density bottle with the dry soil and kerosene covering the soil is placed in a vacuum desiccator, and vacuum is gradually applied. A vacuum of 65 cm is maintained, usually for 16 to 24 hours, till no more air is released from the soil. As far as possible, de-aired kerosene should be used. The bottle is then removed from the desiccator and filled with de-aired kerosene and the stopper is inserted. The bottle is then inserted in a constant temperature ( $27^\circ\text{C}$ ) water-bath for about half an hour. When the bottle has reached the constant temperature, a few drops

of kerosene are added to the bottle (if the room temperature is more than  $27^{\circ}\text{C}$ ) or the extraneous kerosene exuded from the capillary tube is mopped up using a filter paper (if the room temperature is less than  $27^{\circ}\text{C}$ ) as the case may be. The bottle is then wiped dry from outside and weighed. Similarly, before taking mass  $M_4$ , the bottle full of kerosene is kept in the constant temperature bath and then weighed. For accurate determination, three sets of observations are taken and the mean taken to get the final value of  $G$ .

If, however, the test is conducted on any other temperature  $T_i^{\circ}\text{C}$  (*i.e.* room temperature in the case of flask or pycnometer), the specific gravity, to be reported at  $27^{\circ}\text{C}$ , is found as under :

$$G \text{ (at } 27^{\circ}\text{C)} = G \text{ (at } T_i^{\circ}\text{C)} \frac{\text{Sp. gr. of water at } T_i^{\circ}}{\text{Sp. gr. of water at } 27^{\circ}\text{C}} \quad \dots(3.4)$$

Distilled water is used in the flask or pycnometer test, and kerosene is used for the density bottle method. When kerosene is used, its specific gravity is determined accurately by a separate test. The specific gravity of each consignment of kerosene is determined and is repeated monthly or biweekly if the same consignment is still in use.

Table 3.8 shows the observation sheet for the specific gravity determination (See Experiment 2).

**Example 3.1.** *In order to determine the water content, 370 g of a wet sandy sample was placed in a pycnometer. The mass of the pycnometer, sand and water full to the top of the conical cap was found to be 2148 g. The mass of pycnometer full of clean water was 1932 g. Taking  $G = 2.65$ , determine the water content of the sample.*

**Solution.** :

$$\text{From Eq. 3.2,} \quad w = \left[ \frac{M_2 - M_1}{M_3 - M_4} \cdot \frac{G - 1}{G} - 1 \right] \times 100 = \left[ \frac{M}{M_3 - M_4} \cdot \frac{G - 1}{G} - 1 \right] \times 100$$

where  $M = \text{wet mass of soil} = 370 \text{ g}$  ;  $M_3 = 2148 \text{ g}$  ;  $M_4 = 1932 \text{ g}$

$$\therefore w = \left[ \frac{370}{2148 - 1932} \times \frac{2.65 - 1}{2.65} - 1 \right] \times 100 = 6.5\%.$$

**Example 3.2.** *An oven-dried soil having a mass of 200 g is placed in a pycnometer which is then completely filled with water. The total mass of the pycnometer with water and soil inside is 1605 g. The pycnometer filled with water alone has a mass of 1480 g. Calculate the specific gravity of the soil.*

**Solution :** Given :  $M_d = 200 \text{ g}$  ;  $M_3 = 1605 \text{ g}$  ;  $M_4 = 1480 \text{ g}$

$$\therefore G = \frac{M_d}{M_d - (M_3 - M_4)} = \frac{200}{200 - (1605 - 1480)} = 2.67.$$

**Example 3.3.** *The specific gravity of a soil sample was found with the help of a 50 ml density bottle using kerosene. The mass of empty bottle was found to be 62.12 g. A sample of oven dried clay was placed in the bottle and the total mass of bottle and clay was found to be 83.49 g. The mass of bottle, clay and kerosene filled upto the top of the capillary tube of the stopper was found to be 264.41 g. Finally, the bottle full of clean kerosene only has mass of 249.24 g. Every time the temperature of the contents*



of the bottle was maintained at  $27^{\circ}\text{C}$  by placing the bottle in a thermostatically controlled water bath, before taking the weighings. In a separate test, the specific gravity of kerosene was found to be 0.773 at  $27^{\circ}\text{C}$ . What is the specific gravity of soil? If the specific gravity is to be reported at  $4^{\circ}\text{C}$ , what will be its value?

**Solution :** 
$$G = \frac{M_d \times G_k}{M_d - (M_3 - M_4)}$$

Here  $M_d = 83.49 - 62.12 = 21.37 \text{ g}$  ;  $M_3 = 264.41 \text{ g}$  ;  $M_4 = 249.24 \text{ g}$

$M_3 - M_4 = 264.41 - 249.24 = 15.17 \text{ g}$

$\therefore G = \frac{21.37 \times 0.773}{21.37 - 15.17} = \frac{21.37 \times 0.773}{6.2} = 2.66$

Now  $G \text{ at } 4^{\circ}\text{C} = G \text{ at } 27^{\circ}\text{C} \times \frac{\text{Sp. gr. of water at } 27^{\circ}\text{C}}{\text{Sp. gr. of water at } 4^{\circ}\text{C}} = 2.66 \times \frac{0.9965}{1.000} = 2.65$

### 3.4. PARTICLE-SIZE DISTRIBUTION

The percentage of various sizes of particles in a given dry soil sample is found by a particle-size analysis or *mechanical analysis*. By mechanical analysis is meant the separation of a soil into its different size fractions. The mechanical analysis is performed in two stages : (i) sieve analysis and (ii) sedimentation analysis or wet mechanical analysis.

The first stage is meant for coarse-grained soils only, while the second stage is performed for fine-grained soils. In general, a soil sample may contain both coarse-grained particles as well as fine particles, and hence both the stages of the mechanical analysis may be necessary. The sieve analysis is, however, the true representative of grain size distribution, since the test is not affected by temperature etc.

### 3.5. SIEVE ANALYSIS

In the BS and ASTM standards, the sieve sizes are given in terms of the number of openings per inch. The number of openings per square inch is equal to the square of the number of the sieve. In the Indian Standard (IS : 460-1962), the sieves are designated by the size of the aperture in mm. Table 3.1 gives a list of sieves and their openings, for IS, BS and ASTM specifications.

The complete sieve analysis can be divided into two parts — the *coarse analysis* and *fine analysis*. An oven-dried sample of soil is separated into two fractions by sieving it through a 4.75 mm IS sieve. The portion retained on it (+ 4.75 mm size) is termed as the *gravel fraction* and is kept for the coarse analysis, while the portion passing through it (– 4.75 mm size) is subjected to fine sieve analysis. The following set of sieves are used for coarse sieve analysis : IS : 100, 63, 20, 10 and 4.75 mm. The sieves used for fine sieve analysis are : 2 mm, 1.0 mm, 600, 425, 300, 212, 150 and 75 micron IS sieves.

Sieving is performed by arranging the various sieves one over the other in the order of their mesh openings — the largest aperture sieve being kept at the top and the smallest aperture sieve at the bottom. A receiver is kept at the bottom and a cover is kept at the top of the whole assembly. The soil sample is put on the top sieve, and the whole

TABLE 3.1. SIEVES DESIGNATION AND THEIR SIZES

<i>IS Sieves</i> <i>IS : 460-1962</i>		<i>BS Sieves</i> <i>BS : 410-1962</i>		<i>ASTM Sieves</i> <i>ASTM E 11-1961</i>	
<i>Designation</i>	<i>Aperture (mm)</i>	<i>Designation</i>	<i>Aperture (mm)</i>	<i>Designation</i>	<i>Aperture (mm)</i>
50 mm	50.00	2-in	50.80	2-in.	50.80
40 mm	40.00	1½-in.	38.11	1½-in.	38.10
20 mm	20.00	¾-in.	19.05	¾-in.	19.00
10 mm	10.00	⅜-in.	9.52	⅜-in.	9.51
*5.60 mm	5.60	—	—	3½	5.66
4.75 mm	4.75	3/16-in.	4.76	4	4.76
*4.00 mm	4.00	—	—	5	4.00
*2.80 mm	2.80	6	2.80	7	2.83
2.36 mm	2.36	7	2.40	8	2.38
*2.00 mm	2.00	8	2.00	10	2.00
*1.40 mm	1.40	12	1.40	14	1.41
1.18 mm	1.18	14	1.20	16	1.19
*1.00 mm	1.00	16	1.00	18	1.00
710-micron	0.710	22	0.710	25	0.707
600-micron	0.600	25	0.600	30	0.595
*500-micron	0.500	30	0.500	35	0.500
425-micron	0.425	36	0.420	40	0.420
*355-micron	0.355	44	0.355	45	0.354
300-micron	0.300	52	0.300	50	0.297
250-micron	0.250	60	0.250	60	0.250
212-micron	0.212	72	0.210	70	0.210
*180-micron	0.180	85	0.180	80	0.177
150-micron	0.150	100	0.150	100	0.149
*125-micron	0.125	120	0.125	120	0.125
*90-micron	0.090	170	0.090	170	0.088
75-micron	0.075	200	0.075	200	0.074
*63-micron	0.063	240	0.063	230	0.063
*45-micron	0.045	350	0.045	325	0.044

\*Sieves marked with \* have been proposed as an International (ISO) Standard.

It is recommended to include, if possible, these sieves in all sieve analysis data or reports.

assembly is fitted on a sieve shaking machine. The amount of shaking depends upon the shape and the number of particles. At least 10 minutes of shaking is desirable for soils with small particles. The portion of the soil sample retained on each sieve is weighed. The percentage of soil retained on each sieve is calculated on the basis of the total mass

of soil sample taken and from these results, percentage passing through each sieve is calculated. Table 3.14 shows the specimen observation and calculation sheet.

It is advisable to wash the soil portion passing through 4.75 mm sieve over 75 micron sieve so that silt and clay particles sticking to the sand particles may be dislodged. Two grams of sodium hexametaphosphate is added per litre of water used. Washing should be continued until the water passing through 75 micron sieve is substantially clean. The fraction retained on the 75 micron sieve is dried in the oven. The dried portion is then re-sieved through 2 mm, 1 mm, 600, 425, 300, 212, 150 and 75 micron IS sieves. The portion passing 75 micron sieve (while washing) is also dried separately and its mass determined to get % finer than 75 micron size. If the portion passing 75 micron size is substantial, wet analysis is done for further sub-division of particle size distribution. [See Experiment 8].

### 3.6. SEDIMENTATION ANALYSIS : THEORY

In the wet mechanical analysis, or sedimentation analysis, the soil fraction, finer than 75 micron size is kept in suspension in a liquid (usually, water) medium. The analysis is based on Stokes' law, according to which the velocity at which grains settle out of suspension, all other factors being equal, is dependent upon the shape, weight and size of the grain. However, in the usual analysis it is assumed that the soil particles are spherical and have the same specific gravity (the average specific gravity). With this assumption, the coarser particles settle more quickly than the finer ones. If  $v$  is the terminal velocity of sinking of a spherical particle, it is given by

$$v = \frac{2}{9} r^2 \frac{\gamma_s - \gamma_w}{\eta} \quad \dots(3.5)$$

$$\text{or} \quad v = \frac{1}{18} D^2 \frac{\gamma_s - \gamma_w}{\eta} \quad \dots(3.5 \ a)$$

where  $r$  = radius of the spherical particle (m)  
 $D$  = diameter of the spherical particle (m)  
 $v$  = terminal velocity (m/sec)  
 $\gamma_s$  = unit weight of particles ( $\text{kN/m}^3$ )  
 $\gamma_w$  = unit weight of water/liquid ( $\text{kN/m}^3$ )  
 $\eta$  = viscosity of water/liquid ( $\text{kN-s / m}^2$ ) =  $\frac{\mu}{g}$   
 $\mu$  = viscosity in absolute units of poise  
 $g$  = acceleration due to gravity

If water is used as the medium for suspension,  $\gamma_w$  is equal to  $9.81 \text{ kN/m}^3$ . Similarly,  $\gamma_s = G \gamma_w$ . Substituting this, we get

$$v = \frac{1}{18} D^2 \frac{(G - 1) \gamma_w}{\eta} \quad \dots(3.5b)$$

The above formula should be expressed in the consistent units of metres, seconds and kilo-newton. If the diameter ( $D$ ) of the particles is in mm, we have



$$v = \frac{1}{18} \left( \frac{D}{1000} \right)^2 \cdot \frac{(G-1) \gamma_w}{\eta} = \frac{D^2 \cdot \gamma_w (G-1)}{18 \times 10^6 \eta} \quad \dots(3.5 \text{ c})$$

Taking  $\gamma_w = 9.81 \text{ kN/m}^3$ , we get  $v = \frac{D^2 (G-1)}{1.835 \times 10^6 \eta} \quad \dots(3.6)$

$$\therefore D = \sqrt{\frac{18 \times 10^6 \eta v}{(G-1) \gamma_w}} \text{ mm} \quad \dots(3.7)$$

TABLE 3.2. VALUES OF FACTOR F  
(After Calculations by the Author)

Temp. °C.	$\mu$ (Poise)	Factor F					
		G=2.50	G=2.60	G=2.65	G=2.70	G=2.75	G=2.80
15	0.01145	1528	1479	1458	1435	1414	1395
16	0.01116	1508	1460	1438	1417	1396	1377
17	0.01088	1490	1442	1420	1399	1379	1360
18	0.01060	1470	1423	1402	1381	1361	1342
19	0.01034	1452	1406	1384	1364	1344	1325
20	0.01009	1434	1389	1367	1347	1328	1309
21	0.00984	1417	1372	1351	1331	1311	1294
22	0.00961	1400	1355	1335	1315	1296	1278
23	0.00938	1383	1339	1318	1291	1280	1262
24	0.00916	1367	1323	1305	1284	1265	1248
25	0.00896	1351	1308	1288	1269	1251	1233
26	0.00875	1335	1293	1273	1254	1236	1219
27	0.00855	1321	1279	1259	1241	1224	1206
28	0.00836	1305	1264	1244	1226	1208	1191
29	0.00818	1291	1250	1231	1213	1195	1178
30	0.00800	1277	1236	1217	1199	1182	1165
31	0.00783	1264	1224	1205	1187	1170	1154
32	0.00767	1251	1211	1193	1175	1158	1142
33	0.00751	1238	1199	1180	1163	1146	1130
34	0.00736	1225	1186	1168	1151	1134	1118
35	0.00721	1212	1174	1156	1139	1120	1107
36	0.00706	1199	1161	1144	1127	1110	1095
37	0.00692	1188	1150	1133	1116	1100	1084
38	0.00679	1176	1139	1122	1105	1089	1074
39	0.00666	1165	1128	1110	1094	1079	1064
40	0.00654	1155	1118	1101	1084	1069	1055

Note. For intermediate values, interpolation may be done.

or 
$$D = 1355 \sqrt{\frac{\eta v}{G - 1}} \text{ mm} \quad \dots(3.7 a)$$

It should be noted that 1 poise is equivalent to  $0.1 \text{ N} \cdot \text{s}/\text{m}^2$  or to  $10^{-4} \text{ kN} \cdot \text{s}/\text{m}^2$ .

If a particle of diameter  $D$  mm falls through a height  $H_e$  cm in  $t$  minutes,

$$v = \frac{H_e}{60 t} \text{ cm/sec} = \frac{H_e}{6000 t} \text{ m/sec}$$

Substituting in Eq. 3.7, we get

$$D = \sqrt{\frac{18 \times 10^6 \eta}{(G - 1) \gamma_w} \times \frac{H_e}{6000 t}} = \sqrt{\frac{3000 \eta}{(G - 1) \gamma_w}} \cdot \sqrt{\frac{H_e}{t}} \quad \dots(3.8)$$

or 
$$D = 10^{-5} F \sqrt{\frac{H_e}{t}} \quad \dots(3.9)$$

where  $F = 10^5 \sqrt{\frac{3000 \eta}{(G - 1) \gamma_w}}$  is a constant factor for given values of  $\eta$  and  $G$ .

However  $\eta$  depends upon the temperature (see Table 3.2) and hence the factor  $F$  varies with the temperature which changes during the total time of testing. Table 3.2 gives the values of factor  $F$  for various values of  $G$  and temperature. Also, Fig. 3.4 gives the curves for factor  $F$ .

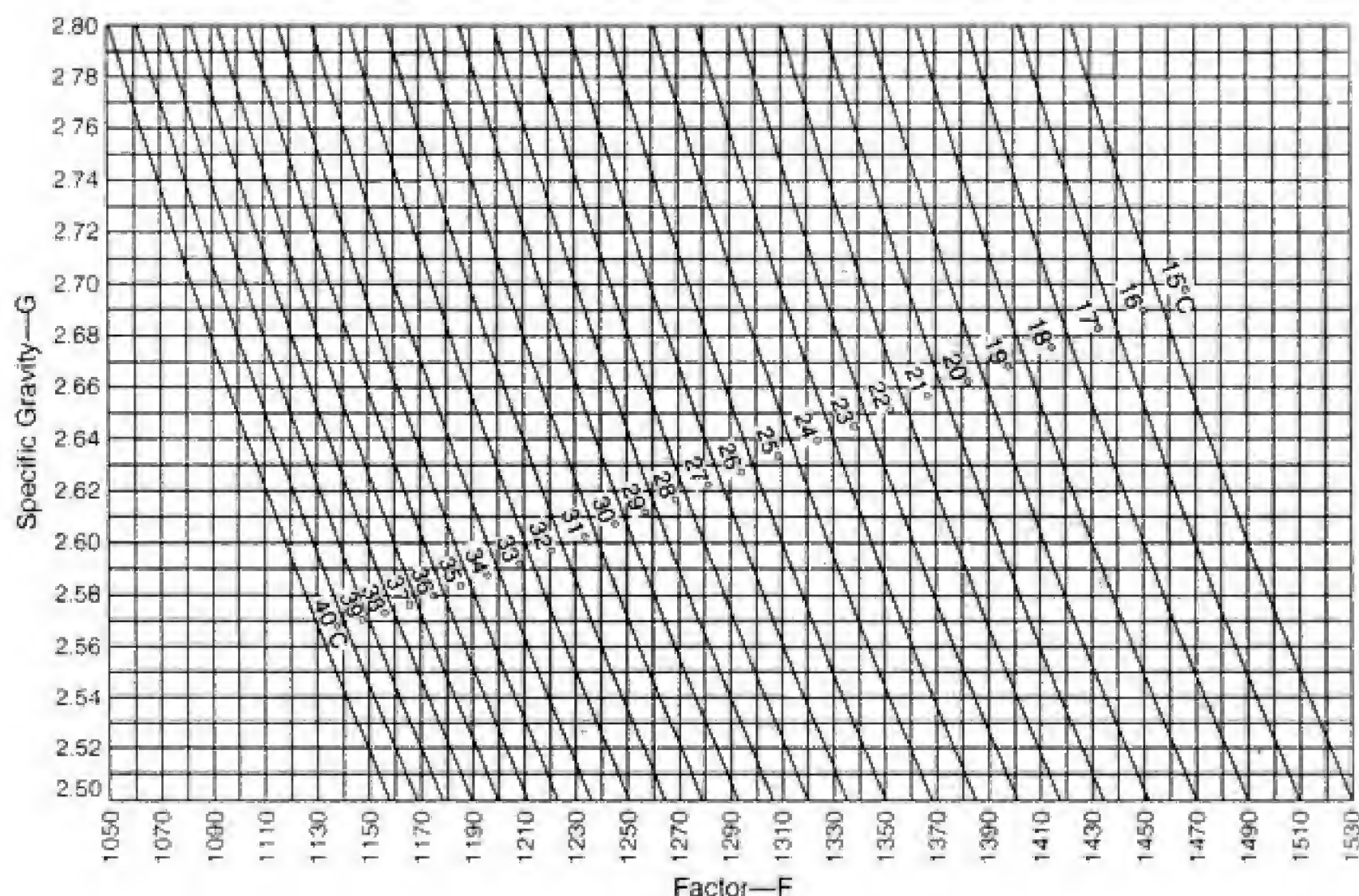
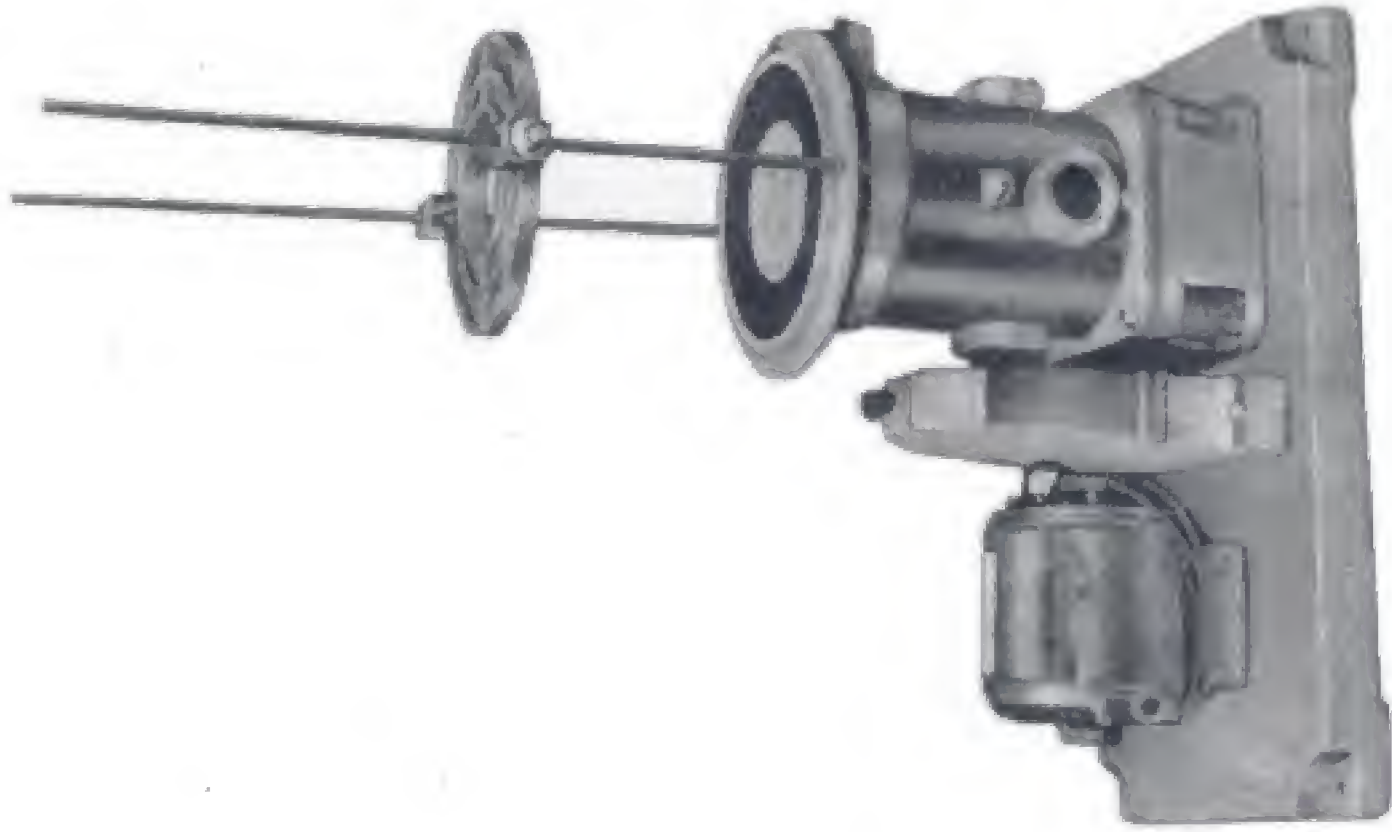


FIG. 3.4. VALUES OF FACTOR  $F$  (AFTER CALCULATION BY THE AUTHOR).

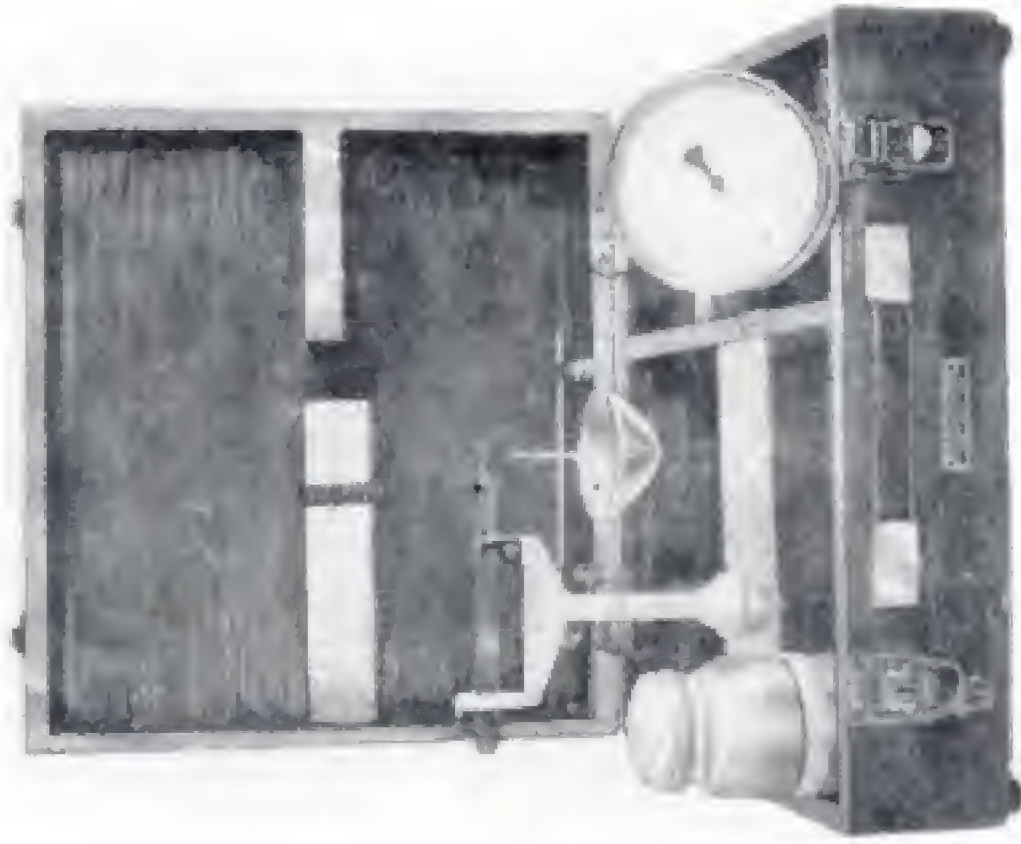




(a) PYCNOMETER (AIMIL)



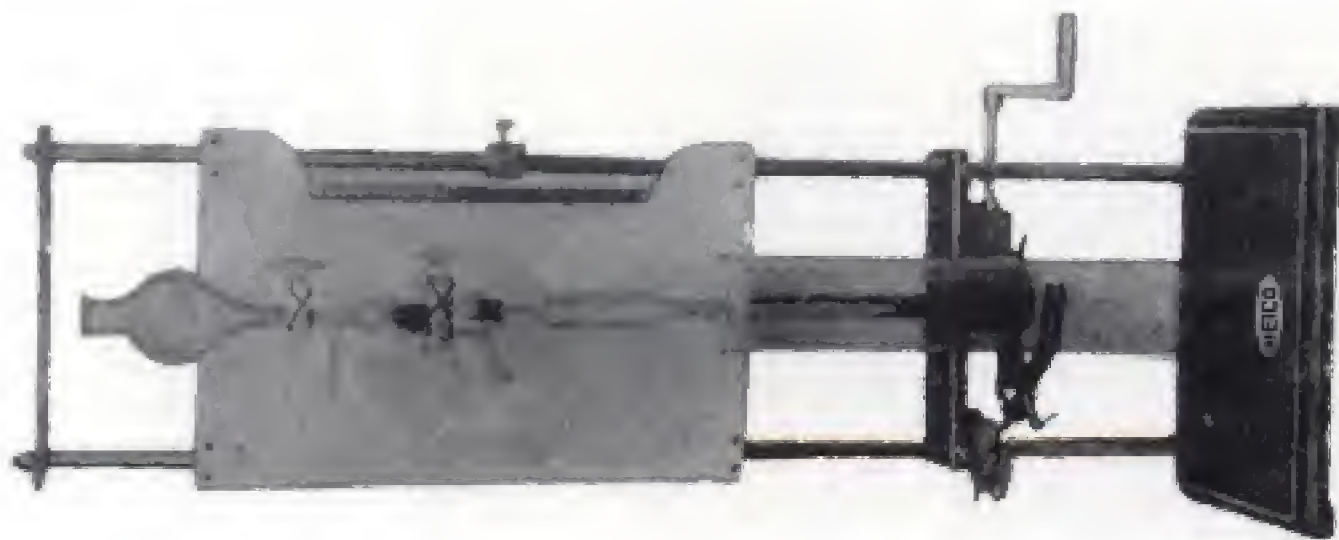
(b) MOTORISED SIEVE SHAKER (AIMIL)



(c) RAPID MOISTURE METER (AIMIL)

PLATE I (AIMIL)

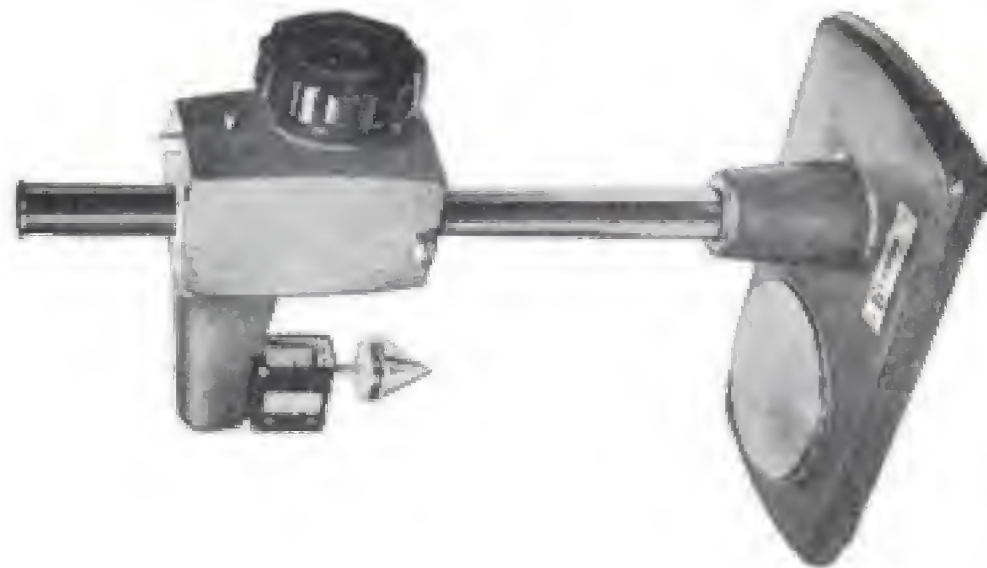




(a) PIPETTE ANALYSIS (AIMIL)



(b) LIQUID LIMIT DEVICE (HEICO)



(c) CONE PENETROMETER (AIMIL/GEONAR)

## PLATE II



(b) SAND REPLACEMENT CYLINDER (HEICO)



(a) PLUMMET BALANCE (AIMIL)

### PLATE III

At 27°C, the viscosity  $\mu$  of distilled water is approximately 0.00855 poise. Since 1 poise is equivalent to  $10^{-4}$  kN-s/m<sup>2</sup>, we have

$$\eta = 0.00855 \times 10^{-4} \text{ kN-s/m}^2$$

Taking an average value of  $G = 2.68$ , we get from Eq. 3.5 (c), we get

$$v = \frac{D^2 (9.81) (2.68 - 1)}{18 \times 10^6 \times 0.0085 \times 10^{-4}} \approx 1.077 D^2 \text{ (m/sec)} \quad \dots(3.10)$$

Eq. 3.10 is an approximate version of Stoke's law and can be easily remembered for rough determinations. Based on this, the time of settlement of particles of various diameters, through a height of 10 cm are as under :

Diameter (mm)	Time
0.06	25.8 s
0.02	3 m 52 s
0.01	15 m 28 s
0.006	42 m 59 s
0.004	1 h 36 m 43 s
0.002	6 h 26 m 53 s
0.001	25 h 47 m 31 s

The sedimentation analysis is done either with the help of a hydrometer or a pipette. In both the methods, a suitable amount of oven-dried soil sample, finer than 75 micron size, is mixed with a given volume  $V$  of distilled water. The mixture is shaken thoroughly and the test is started by keeping the jar, containing soil water mixture, vertical. At the commencement of sedimentation test, soil particles are assumed to be uniformly distributed throughout the suspension. After any time interval  $t$ , if a sample of soil suspension is taken from a height  $H_e$  (measured from the top level of suspension), only those particles will remain in the suspension which have not settled during this time interval. The diameter of those particles, which are finer than those which have already settled, can be found from Eq. 3.9. The greater the time interval  $t$  allowed for suspension to settle, the finer are the particles sizes retained at this depth  $H_e$ . Hence sampling at different time intervals, at this sampling depth  $H_e$ , would give the content of particles of different sizes.

If, at any time interval  $t$ ,  $M_D$  is the mass, per ml, of all particles smaller than the diameter  $D$  (determined from Eq. 3.9) still in suspension at the depth  $H_e$  the percentage finer than  $D$  is given by

$$N = \frac{M_D}{M_d/V} \times 1000 \quad \dots(3.11)$$

where  $N$  = percentage finer than the diameter  $D$

$M_d$  = total dry mass of all particles put in the suspension

$V$  = volume of suspension.

Thus, with the help of Eqs. 3.9 and 3.11, we can get various diameter  $D$  and the percentage of particles finer ( $N\%$ ) than this diameter.



**Limitations of sedimentation analysis.** The analysis is based on the assumptions that (i) soil particles are spherical, (ii) particles settle independent of other particles and the neighbouring particles do not have any effect on its velocity of settlement and (iii) the walls of jar, in which the suspension is kept, also do not affect the settlement. In actual practice, the fine particles of soil, for which this analysis is primarily meant, are not truly spherical. The particles of fine grained soils are thin platelets which do not settle out of suspension in the same manner and at the same rate as smooth spheres. Thus, the sedimentation analysis gives the particles size *equivalent diameter*. The upper limit of particle size for the validity of the law is about 0.2 mm (Taylor : 1948) beyond which the liquid tends to develop a turbulent motion at the boundaries of the particles. The lower limit of particle size is about 0.0002 mm. For particle smaller than 0.0002 mm equivalent diameter, Brownian movement affects their settlement, and Stoke's law no longer remains valid. Also it is assumed that the soil has an average specific gravity, the value of which is used in computing the diameter  $D$ . Actually, different particles may have different specific gravity, depending upon their mineral constituents. The settlement of the particles is influenced by the surrounding particles as the liquid is not of infinite extent. The particles falling near the wall of the jar are also affected.

### 3.7. PIPETTE METHOD

The pipette method is the standard sedimentation method used in the laboratory. The equipment consists of a pipette, a jar and a number of sampling bottles. Generally, a boiling tube of 500 ml capacity is used in place of a jar. Fig. 3.5 shows a pipette for extracting samples from the jar (or tube) from a desired depth ( $H_s$ ). The pipette consists of (i) a 125 ml bulb with stop cock, for keeping distilled water, (ii) a three way stop cock, (iii) suction and waste water outlets and (iv) sampling pipette of 10 ml capacity (including the capacity of the cock). The method consists in drawing off samples of soil suspension, 10 ml in volume, by means of this pipette from a depth of 10 cm ( $H_s$ ) at various time intervals after the commencement of the sedimentation. The recommended time intervals are : 1/2, 1, 2, 4, 8, 15 and 30 minutes, and 1, 2, 4, 8, 16 and 24 hours, reckoned from the commencement of the test. The pipette should be inserted in the boiling tube about 25 seconds before the selected time interval and the time taken for sucking the sample should not be more than 10 to 20 seconds. Each sample, so taken, is transferred into suitable sampling bottles and dried in an oven. The mass  $M_D$  of solids per ml of suspension is thus found by taking the dry mass and dividing it by 10.

**Method of preparing soil suspension.** In the sedimentation analysis, only those particles which are finer than 75 micron size are included. Hence the soil sample is washed through a 75 micron sieve. About 12 to 30 g of oven-dried sample (depending upon the type of soil) is accurately weighed and mixed with distilled water in a dish or beaker to form a smooth thin paste. To have proper dispersion of soil, a dispersing agent (deflocculating agent) is added to the soil. Some of the common dispersing agents are : sodium oxalate, sodium silicate and sodium polyphosphate compounds, such as tetra sodium pyrophosphate, sodium hexametaphosphate (*calgon*) and sodium tripolyphosphate. IS : 2720 (Part IV)-1965 recommends the use of dispersing solution containing 33 g of the sodium-hexametaphosphate

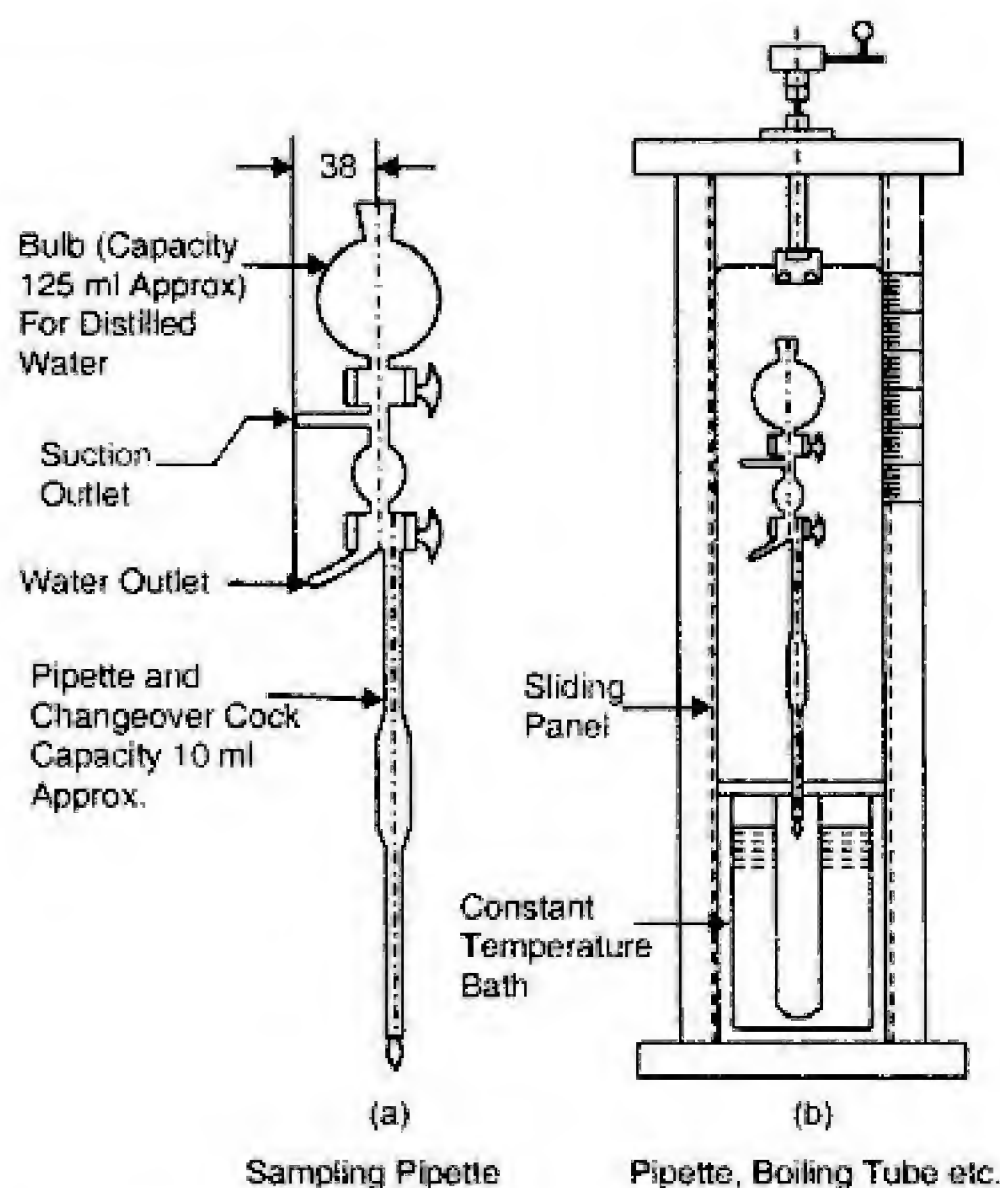


FIG. 3.5. PIPETTE ANALYSIS APPARATUS.

and 7 g of sodium carbonate in distilled water to make one litre of solution. 25 ml of this solution is added to the dish (containing the soil and distilled water) and the mixture is warmed gently for about 10 minutes. The contents are then transferred to the cup of a mechanical mixer, using a jet of distilled water to wash all traces of the soil out of the evaporating dish. The soil suspension is then stirred well for 15 minutes or longer in the case of highly clayey soils. The suspension is then washed through 75 micron sieve, using jet of distilled water and the suspension, which has passed through the sieve, is transferred to the 500 ml capacity boiling tube (sedimentation tube). Care should be taken that all the particles finer than 75 micron size are transferred to the tube. The tube is then filled to the 500 ml mark, by adding distilled water. The tube is then put in a constant temperature water bath [Fig. 3.5 (b)], where used. When the temperature in the tube has been stabilised to the temperature of the bath, the soil suspension is thoroughly shaken by inverting the tube several times, and then replaced in the bath. The stop watch is then started, and soil samples are collected at various time intervals, with the help of pipette.

Those soils, which contain organic matter and calcium compounds, are *pretreated* before the dispersing agent is mixed (as explained above) since these contents act as cementing agents and cause the particles to settle as aggregations of particles instead of as individuals. The process of removal of organic matter and calcium compounds is known as *pretreatment*.

The soil is first treated with hydrogen peroxide solution to remove the organic matter by oxidation. The mixture of soil and hydrogen peroxide is kept warm at a temperature not exceeding 60 °C, till no further evolution of the gas takes place. The remaining hydrogen peroxide in the solution is then decomposed by boiling the solution. To remove the calcium compound, the cooled mixture of soil is then treated with 0.2N hydrochloric acid. When the reaction is complete, the mixture is filtered and the filtrate is washed with distilled water until it becomes free from the acid. The filtrate is then dried in the oven, to know the loss of mass due to pretreatment.

#### Calculation of $D$ and $N$

10 ml samples are collected from the soil suspension (sedimentation tube) from a depth of 10 cm, with the help of the pipette, at various time intervals. The samples are collected into the weighing bottles (sampling bottles), and kept in the oven for drying. The mass  $M_D$ , per ml of suspension so collected is calculated as under :

$$M_D = \frac{\text{Dry mass of sample in the weighing bottle}}{V_p} \quad \dots(3.12)$$

where  $V_p$  = volume of the pipette  
 = volume of sample collected in the weighing bottle = 10 ml.

The percentage finer is calculated from the following expression based on Eq. 3.11:

$$N' = \frac{M_D - \frac{m}{V}}{M_d/V} \times 100 \quad \dots(3.13)$$

where  $m$  = mass of dispersing agent present in the total suspension of volume  $V$   
 $V$  = volume of suspension = 500 ml  
 $N'$  = percentage finer, based on  $M_d$

If 25 ml of dispersing agent solution, containing 33 g of sodium hexametaphosphate and 7 g of sodium carbonate per litre is used, we get

$$m = \frac{33 + 7}{1000} \times 25 = 1 \text{ g}$$

The corresponding diameter  $D$  of the particle, to which the above percentage of soil is finer, is calculated from Eq. 3.9. The observation may be recorded as shown in Table 3.16. The pipette method, though very simple, requires more time, and is not suitable for routine control tests. The apparatus is very sensitive, and very accurate weighings are required. Due to these reasons, sometimes the hydrometer method of sedimentation analysis is preferred.

### 3.8. HYDROMETER METHOD

The hydrometer method of sedimentation analysis differs from the pipette analysis in the method of taking the observations — the principles of the test being the same in both the cases. In the pipette analysis, the mass  $M_D$  per ml of suspension is found directly by collecting a 10 ml sample of soil suspension from the sampling depth  $H_c$ . However, in the hydrometer analysis,  $M_D$  is computed indirectly by reading the density of the soil



suspension at a depth  $H_e$  at various time intervals. In the pipette test, the sampling depth  $H_e$  is kept constant ( $= 10$  cm) while in the hydrometer test, the sampling depth  $H_e$  (also known as the effective depth) goes on increasing as the particles settle with the increase in the time intervals. It is, therefore, necessary to calibrate the hydrometer and the sedimentation jar before the start of the sedimentation test, to find a relation between  $H_e$  and the density readings of the hydrometer. If the same set of hydrometer and sedimentation jar are used for a number of tests, one calibration chart will serve the purpose of the tests.

**Calibration of hydrometer.** Fig. 3.6 (a) shows the hydrometer. The readings on the hydrometer stem give the density of the soil suspension situated at the centre of the bulb at any time. For convenience, the hydrometer readings are recorded after subtracting 1 and multiplying the remaining digits by 1000. Such a *reduced reading* is designated as  $R_h$ . For example, if the density reading at the intersection of horizontal surface of soil suspension with the stem, is 1.010, it is recorded as 10 (i.e.,  $R_h = 10$ ). Similarly, a density reading of 0.995 is recorded as  $R_h = -5$ . As indicated in Fig. 3.6(a) the hydrometer readings  $R_h$  (and also the density readings) increase in the downward direction towards the hydrometer bulb. Let  $H$  be the height, in cm, between any hydrometer reading  $R_h$  and the neck, and  $h$  the height of the bulb.

Fig. 3.6(b) shows the jar, containing the soil suspension. When the hydrometer is immersed in the jar [Fig. 3.6 (c)], the water level  $aa$  rises to  $a_1a_1$ , the rise being equal

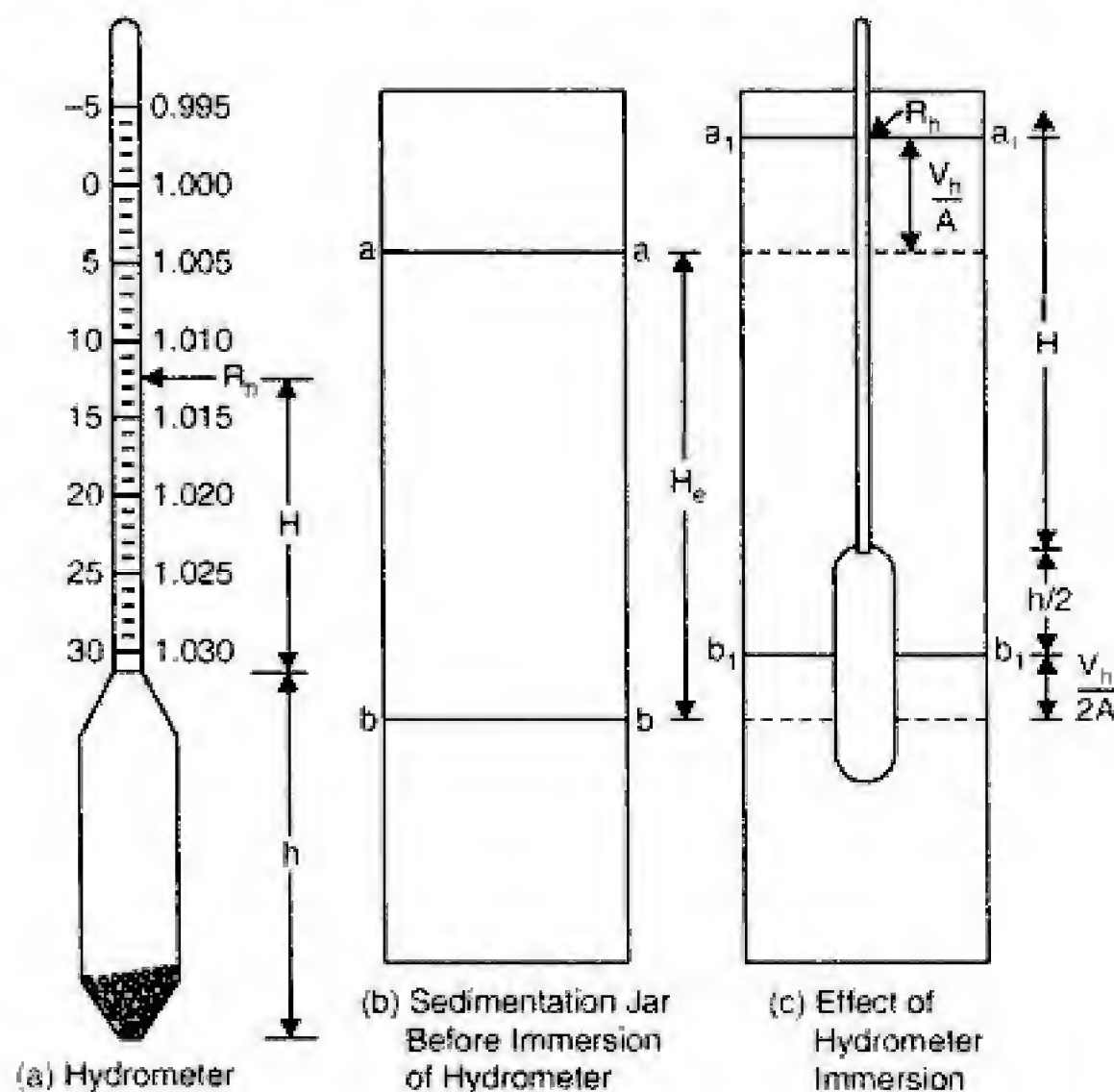


FIG. 3.6. HYDROMETER ANALYSIS.

to the volume  $V_h$  of the hydrometer divided by the internal area of cross-section  $A$  of the jar. Similarly, the level  $bb$  rises to  $b_1b_1$ , where  $bb$  is the level, situated at a depth  $H$ , below the top level  $aa$ , at which the density measurements of the soil suspension are being taken. The rise between  $bb$  and  $b_1b_1$  will be approximately equal to  $V_h/2A$ . The level  $b_1b_1$  is now corresponding to the centre of the bulb, but the soil particles at  $b_1b_1$  are of the same concentration as they were at  $bb$ . Therefore, we have

$$H_e = \left( H + \frac{h}{2} + \frac{V_h}{2A} \right) - \frac{V_h}{A} = H + \frac{1}{2} \left( h - \frac{V_h}{A} \right) \quad \dots(3.14)$$

In the above expression, there are two variables : the effective depth  $H_e$  and the depth  $H$  which depends upon the hydrometer reading  $R_h$ . Therefore, by selecting various hydrometer readings  $R_h$ , the depth  $H$  can be measured with the help of an accurate scale, and the corresponding depth  $H_e$  can be found. The height  $h$  of the bulb is constant. Similarly,  $V_h$  and  $A$  are constant. To find the volume of the hydrometer, it is weighed accurately. The mass of the hydrometer in grams gives the volume of the hydrometer in millilitres. The observations are tabulated as shown in Table 3.3.

TABLE 3.3 CALIBRATION OF HYDROMETER

1. Hydrometer No. 25	1. Sedimentation Jar No. 5.
2. Volume of Hydrometer $V_h = 72\text{cm}^3$	2. Sectional Area of Jar $A = 30\text{ cm}^2$
3. Height of bulb ( $h$ ) = 16.6 cm	3. Constant $\frac{1}{2} \left( h - \frac{V_h}{A} \right) = 7.1\text{ cm.}$

Hydrometer reading $R_h$	$H$ (cm)	Effective depth $H_e$ (cm)
30	1.8	8.9
25	3.6	10.7
20	5.4	12.5
15	7.3	14.4
10	9.2	16.3
5	11.1	18.2
0	13.0	20.1
-5	14.9	22.0

Based on the observations of Table 3.3, a calibration curve (straight line) has been shown in Fig. 3.7. The curve is useful in finding the value of  $H_e$  for any value of hydrometer reading  $R_h$ .

**Test procedure.** The method of preparation of soil suspension is the same as indicated in the pipette test. However, the volume of suspension is 1000 ml in this case and hence, double the quantity of dry soil and dispersing agent is taken. The sedimentation jar (cylinder) is shaken vigorously and is then kept vertical over a solid base. The stop watch is simultaneously started. The hydrometer is slowly inserted in the jar and readings are taken at 1/2, 1 and 2 minutes time intervals. The hydrometer is then taken out. More readings are then taken at the following time intervals : 4, 8, 15, 30 minutes and 1, 2, 4 hours etc. To take the reading, the hydrometer is inserted about 30 seconds before the given

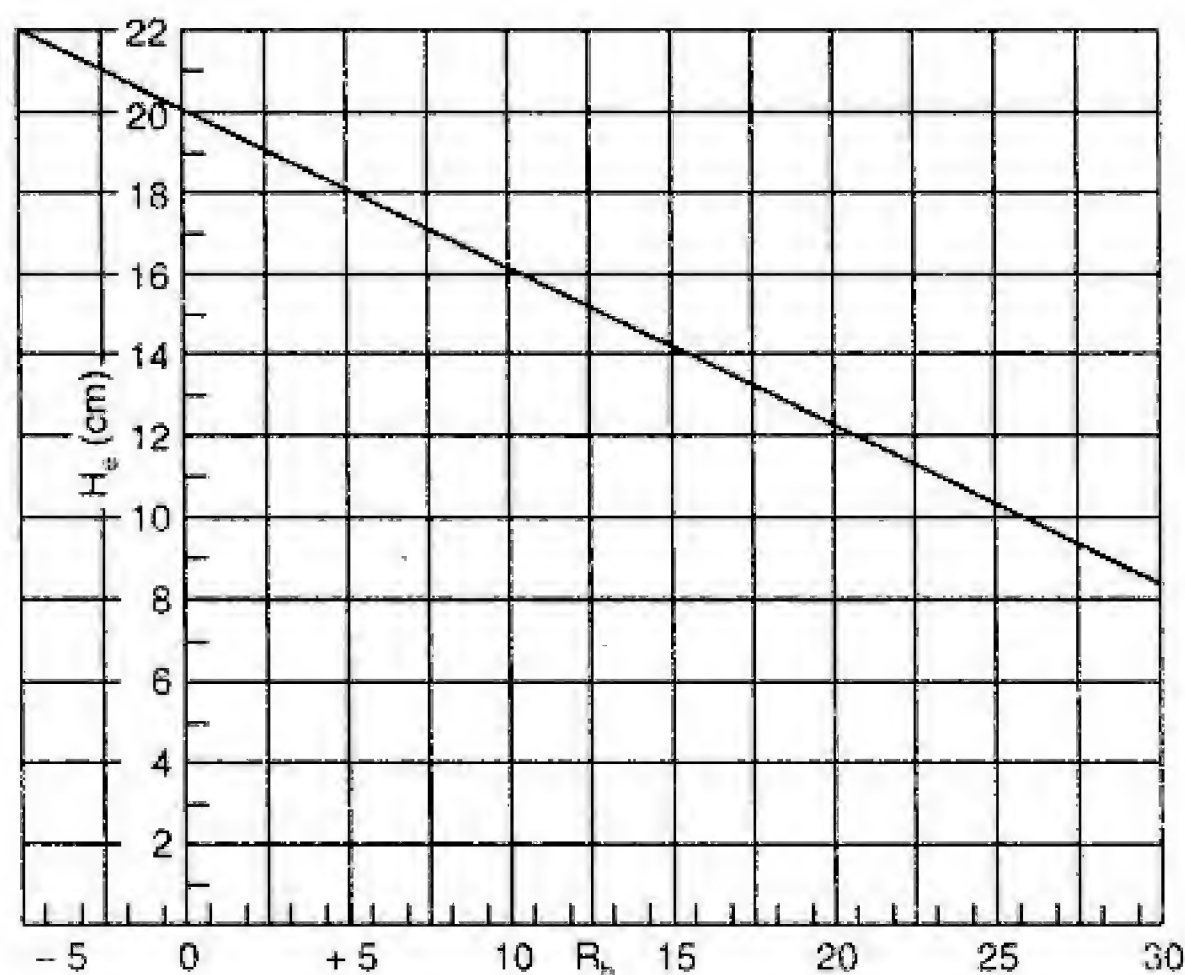


FIG. 3.7. CALIBRATION CURVE FOR THE HYDROMETER.

time interval, so that it is stable at the time when the reading is to be taken. Since the soil suspension is opaque, the reading is taken corresponding to the upper level of the meniscus. A suitable meniscus correction is then applied to the hydrometer readings.

**Corrections to the hydrometer readings.** The hydrometers are generally calibrated at  $27^{\circ}\text{C}$ . If the temperature of the soil suspension is not  $27^{\circ}\text{C}$ , a *temperature correction*  $C_t$  should be applied to the observed hydrometer reading. If the test temperature is more than  $27^{\circ}\text{C}$ , the hydrometer readings will naturally be less than what they should be, and hence the temperature correction will be *positive*. Similarly, if the test temperature is lower than  $27^{\circ}$ , the temperature correction will be *negative*. In addition to this, two more corrections are to be applied to the hydrometer readings: the *meniscus correction* and the *dispersing agent correction*. Since the soil suspension is opaque, the hydrometer reading is taken at the top of the meniscus. Actual reading, to be taken at the water level, will be *more* since the readings increase in the downward direction. Hence the meniscus correction  $C_m$  is always *positive*. Its magnitude can be found by immersing hydrometer in a jar containing clear water, and finding the difference between the reading corresponding to the top and bottom of the meniscus. Similarly, the addition of dispersing agent in water increase its density, and hence the *dispersing agent correction*  $C_d$  is always *negative*. Thus, the corrected hydrometer reading  $R$  is given by

$$R = R_h' + C_m \pm C_t - C_d \quad \dots(3.15)$$

where  $R_h'$  = observed hydrometer reading at the top of the meniscus.

The three corrections can be combined into one correction, known as the *composite correction*  $\pm C$  and Eq. 3.15 can be represented as



$$R = R_h' \pm C \quad \dots(3.16)$$

Similarly, the hydrometer reading, corrected for meniscus above, is given by

$$R_h = R_h' + C_m \quad \dots(3.17)$$

Eq. 3.17 is useful for finding the effective height  $H_e$  from the calibration chart (curve) corresponding to the observed reading  $R_h'$ .

In order to find the composite correction  $C$ , an identical cylinder with 100 ml capacity is taken, and filled with distilled water and the same quantity of dispersing agent is used in the test cylinder. The temperature of both the cylinders should be the same. The hydrometer is immersed in this comparison cylinders containing distilled water and dispersing agent, and reading is taken at the *top* of the meniscus. The *negative* of the hydrometer reading so obtained gives the composite correction. For example, if the hydrometer reading in the comparison cylinder is +2, the composite correction  $C = -2$ . Similarly, if the reading is -1, the composite correction will be  $C = +1$ . The composite correction is found before the start of the test, and also, at every time interval exceeding 30 minutes.

**Computation of  $D$  and  $N$ .** The particle size  $D$  is calculated from Eq. 3.9:

$$D = 10^{-5} F \sqrt{\frac{H_e}{t}} \quad \dots(3.9)$$

For various time intervals,  $R_h$  is found from Eq. 3.7 and corresponding values of  $H_e$  from Fig. 3.7. Substituting the values of  $H_e$  (cm) and  $t$  (minutes) in Eq. 3.9, the diameter  $D$  (mm) is computed. To compute the percentage of the soil finer than this diameter, the mass  $M_D$  per ml of suspension at effective depth  $H_e$  is first computed as under. Since the hydrometer readings have been recorded by subtracting 1 from the density ( $\rho$ ) readings and multiplying them by 1000, we have

$$R = (\rho - 1) 1000 \quad \text{or} \quad \rho = 1 + \frac{R}{1000} \quad \dots(i)$$

where  $\rho$  is the density reading actually marked on the hydrometer, and  $R$  is the hydrometer reading corrected for the composite correction.

Now let us consider 1 ml of soil suspension, at a time interval  $t$ , at the effective depth  $H_e$ . If  $M_D$  is the mass of solids in this 1 ml suspension, the mass of water

in it will be  $= 1 - \frac{M_D}{G}$

Total mass of 1 ml suspension  $= 1 - \frac{M_D}{G} + M_D$

Hence density of the suspension  $= 1 - \frac{M_D}{G} + M_D \quad \dots(ii)$

Equating (i) and (ii), we get  $1 + \frac{R}{1000} = 1 - \frac{M_D}{G} + M_D$

$$\text{or} \quad M_D = \frac{R}{1000} \left( \frac{G}{G-1} \right) \quad \dots(3.18)$$

where  $G$  = specific gravity of soil solids (average)

Substituting this values of  $M_D$  in Eq. 3.11, we get

$$N' = \frac{R}{\frac{1000}{M_d/V} \left( \frac{G}{G-1} \right)} \times 100 \quad ; \text{ Taking } V=1000 \text{ ml, we get}$$

$$N' = \frac{100 G}{M_d (G-1)} R \quad \dots(3.19)$$

where  $N'$  = percentage finer with respect to  $M_d$

Thus, for various values of  $R$ ,  $N'$  can be computed. For a combined sieve and sedimentation analysis, if  $M$  is the total dry mass of soil originally taken (before sieving it over 2 mm sieve), the overall percentage finer  $N$  is given by

$$N = N' \times \frac{M'}{M} \quad \dots(3.20)$$

where  $M'$  = cumulative mass passing 2 mm sieve (out of which the soil having mass  $M_d$  was taken for the wet analysis)  
 $M$  = total dry mass of soil sample

If the soil sample does not contain particles coarser than 2 mm size,  $N$  and  $N'$  will be equal. Table 3.15 gives the observation sheet for hydrometer analysis [See Experiment 9].

### 3.9. PARTICLE SIZE DISTRIBUTION CURVE

The results of the mechanical analysis are plotted to get a particle-size distribution curve with the percentage finer  $N$  as the ordinate and the particle diameter as the abscissa, the diameter being plotted on a logarithmic scale. Fig. 3.8 shows some typical curves for various soils. A particle-size distribution curve gives us an idea about the *type* and *gradation* of the soil. A curve situated higher up or to the left represents a relatively fine grained soil while a curve situated to the right represents a coarse grained soil.

A soil sample may be either *well graded* or *poorly graded (uniformly graded)*. A soil is said to be well graded when it has good representation of particles of all sizes. On the other hand, a soil is said to be poorly graded if it has an excess of certain particles and deficiency of other, or if it has most of the particles of about the same size ; in the latter case it is known as a uniformly graded soil. Thus, soil *A* (Fig. 3.8) is well graded while soil *B* is uniformly graded. A curve with a flat portion represent a soil in which some intermediate size particles are missing (soil *E* in Fig. 3.8). Such a soil is known as *gap graded* or *skip graded*.

For coarse grained soil, certain particle sizes such as  $D_{10}$ ,  $D_{30}$  and  $D_{60}$  are important. The  $D_{10}$  represents a size, in mm such that 10% of the particles are finer than this size. Similarly, the soil particles finer than  $D_{60}$  size are 60 per cent of the total mass of the sample. The size  $D_{10}$  is sometimes called the *effective size* or *effective diameter*. The *uniformity coefficient*  $C_u$  (or coefficient of uniformity) is a measure of particle-size range and is given by the ratio of  $D_{60}$  and  $D_{10}$  sizes :

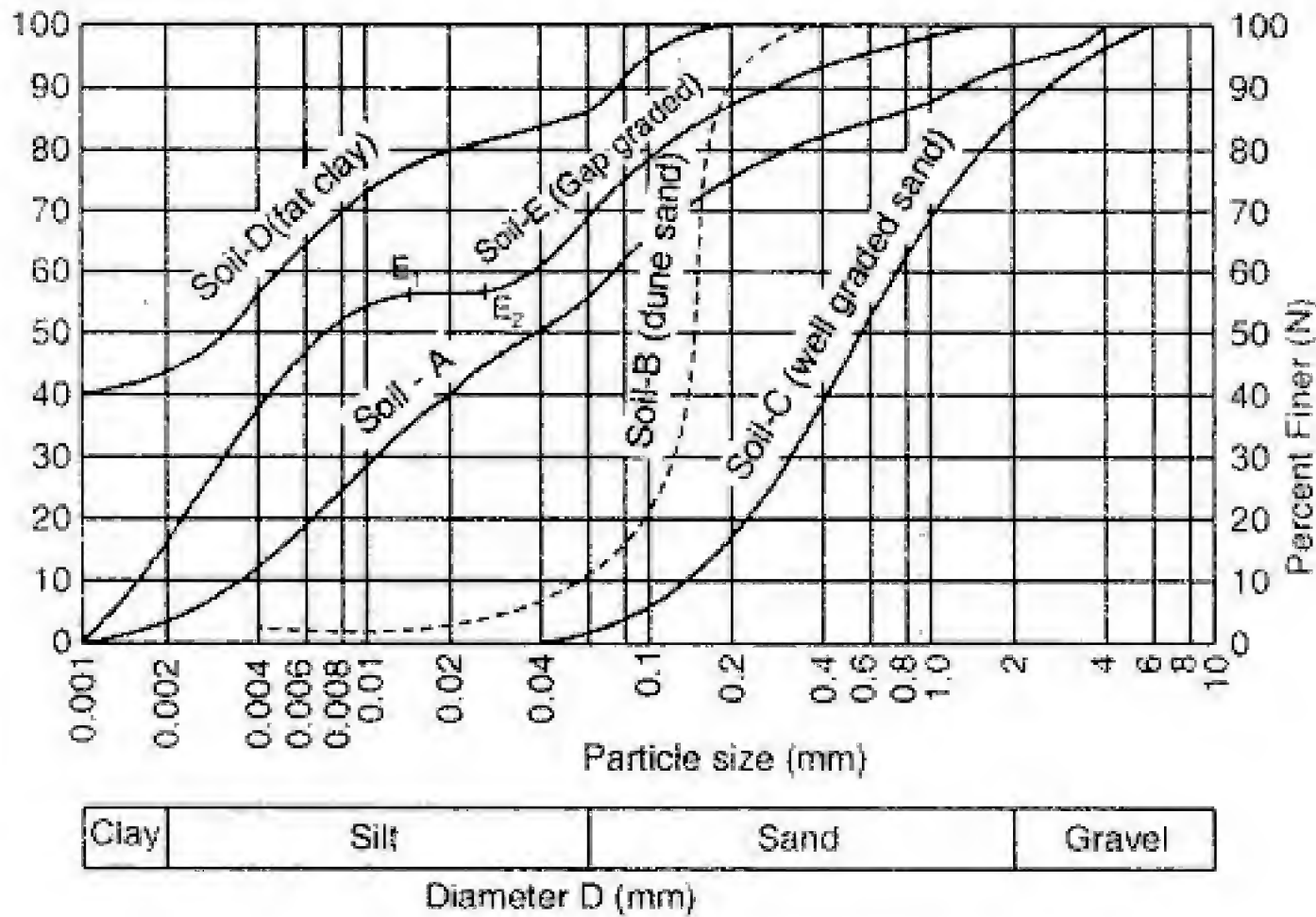


FIG. 3.8. PARTICLE SIZE DISTRIBUTION CURVE

$$C_u = \frac{D_{60}}{D_{10}} \quad \dots(3.21)$$

Similarly, the shape of the particle size curve is represented by the *co-efficient of the curvature*  $C_c$ , given by

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \quad \dots(3.22)$$

For a uniformly graded soil,  $C_u$  is nearly unity. For a well graded soil,  $C_c$  must be between 1 to 3 and in addition  $C_u$  must be greater than 4 for gravels and 6 for sands (USBR : 1960).

**Example 3.4.** A soil sample, consisting of particles of size ranging from 0.5 mm to 0.01 mm, is put on the surface of still water tank 5 metres deep. Calculate the time of settlement of the coarsest and the finest particles of the sample, to the bottom of the tank. Assume average specific gravity of soil particles as 2.66 and viscosity of water as 0.01 poise.

**Solution.** 
$$v = \frac{D^2 \gamma_w (G - 1)}{18 \times 10^6 \eta} = \frac{D^2 (G - 1)}{1.835 \times 10^6 \eta} \quad \dots(3.6)$$

Here,  $G = 2.66$  and  $\eta = 0.01 \times 10^{-1} = 10^{-6} \text{ kN-s/m}^2$

$$\therefore v = \frac{D^2}{1.835} \times \frac{2.66 - 1}{10^6 (10^{-6})} \approx 0.905 D^2$$

where  $v$  is in m/sec and  $D$  is in mm.



For coarsest particle,  $D = 0.5 \text{ mm}$  ;  $v = 0.905 (0.5)^2 = 0.2263 \text{ m/sec}$

$$\therefore t = \frac{h}{v} = \frac{5}{0.2263} = 22.1 \text{ seconds}$$

For the finest particle,  $D = 0.01 \text{ mm}$  ;  $v = 0.905 (0.01)^2 = 9.05 \times 10^{-5} \text{ m/sec}$

$$\therefore t = \frac{5}{9.05 \times 10^{-5}} = 55249 \text{ sec} = 15 \text{ hours } 20 \text{ min } 49 \text{ sec.}$$

**Example 3.5.** 50 grams of oven dried soil sample is taken for sedimentation analysis. The hydrometer reading in a 100 ml soil suspension 30 minutes after the commencement of sedimentation test is 24.5. The effective depth for  $R_h = 25$ , found from the calibration curve is 10.7 cm. The meniscus correction is found to be + 0.5 and the composite correction as - 2.50 at the test temperature of 30 °C. Taking the specific gravity of particles as 2.75 and viscosity of water as 0.008 poise, calculate the smallest particle size which would have settled during this interval of 30 minutes and the percentage of particles finer than this size.

**Solution.**  $R_h' = 24.5$  ;  $\therefore R_h = 24.5 + 0.5 = 25$  ;  $R = 24.5 - 2.50 = 22$

From Eq. 3.8, 
$$D = \sqrt{\frac{3000 \eta}{(G - 1) \gamma_w}} \cdot \sqrt{\frac{H_e}{t}}$$

where  $D$  is in mm,  $H_e$  is in cm and  $t$  is in min.

For the present case,  $\eta = 0.008 \times 10^{-4} \text{ kN-s/m}^2$ ,  $H_e = 10.7 \text{ cm}$ ,  $G = 2.75$  and  $\gamma_w = 9.81 \text{ kN/m}^3$  ;  $t = 30 \text{ min}$ .

$$\therefore D = \sqrt{\frac{3000 \times 0.008 \times 10^{-4}}{(2.75 - 1) 9.81}} \times \sqrt{\frac{H_e}{t}} = 0.01182 \sqrt{\frac{H_e}{t}}$$

or 
$$D = 0.01182 \sqrt{\frac{10.7}{30}} = 7.06 \times 10^{-3} \text{ mm} = 0.00706 \text{ mm}$$

Alternatively, the diameter could be found from the expression  $D = 10^{-5} F \sqrt{\frac{H_e}{t}}$

From Table 3.2, for  $G = 2.75$  and temperature = 30° C, we get  $F = 1182$

$$\therefore D = 10^{-5} \times 1182 \sqrt{\frac{10.7}{30}} = 0.00706 \text{ mm.}$$

The percentage finer is given by

$$N = \frac{100 G}{M_d (G - 1)} R \quad \text{where } M_d = \text{mass of dry soil} = 50 \text{ g}$$

$$\therefore N = \frac{100 \times 2.75}{50 (2.75 - 1)} \times 22 = 69.1 \%$$

### 3.10. CONSISTENCY OF SOILS

By *consistency* is meant the relative ease with which soil can be deformed. This term is mostly used for fine grained soils for which the consistency is related to a large extent to water content. Consistency denotes degree of firmness of the soil which may

be termed as *soft, firm, stiff* or *hard*. Fine grained soil may be mixed with water to form a plastic paste which can be moulded into any form by pressure. The addition of water reduces the cohesion making the soil still easier to mould. Further addition of water reduces the cohesion until the material no longer retains its shape under its own weight, but flows as a liquid. Enough water may be added until the soil grains are dispersed in a suspension. If water is evaporated from such a soil suspension, the soil passes through various stages or *states of consistency*. In 1911, the Swedish agriculturist Atterberg divided the entire range from liquid to solid state into four stages: (i) the liquid state, (ii) the plastic state, (iii) the semi-solid state and (iv) the solid state. He set arbitrary limits, known as *consistency limits* or *Atterberg limits*, for these divisions in terms of water content. Thus, the consistency limits are the water contents at which the soil mass passes from one state to the next. Fig. 3.9 shows the four states of consistency, with the appropriate consistency limits. The Atterberg limits which are most useful for engineering purposes are: *liquid limit, plastic limit* and *shrinkage limit*. These limits are expressed as per cent water content.

**Liquid limit ( $w_L$ ):** Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil. It is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing which can be measured by standard available means. With reference to the standard liquid limit device, it is defined as the minimum water content at which a part of soil cut by a groove of standard dimensions, will flow together for a distance of 12 mm ( $\frac{1}{2}$  inch) under an impact of 25 blows in the device.

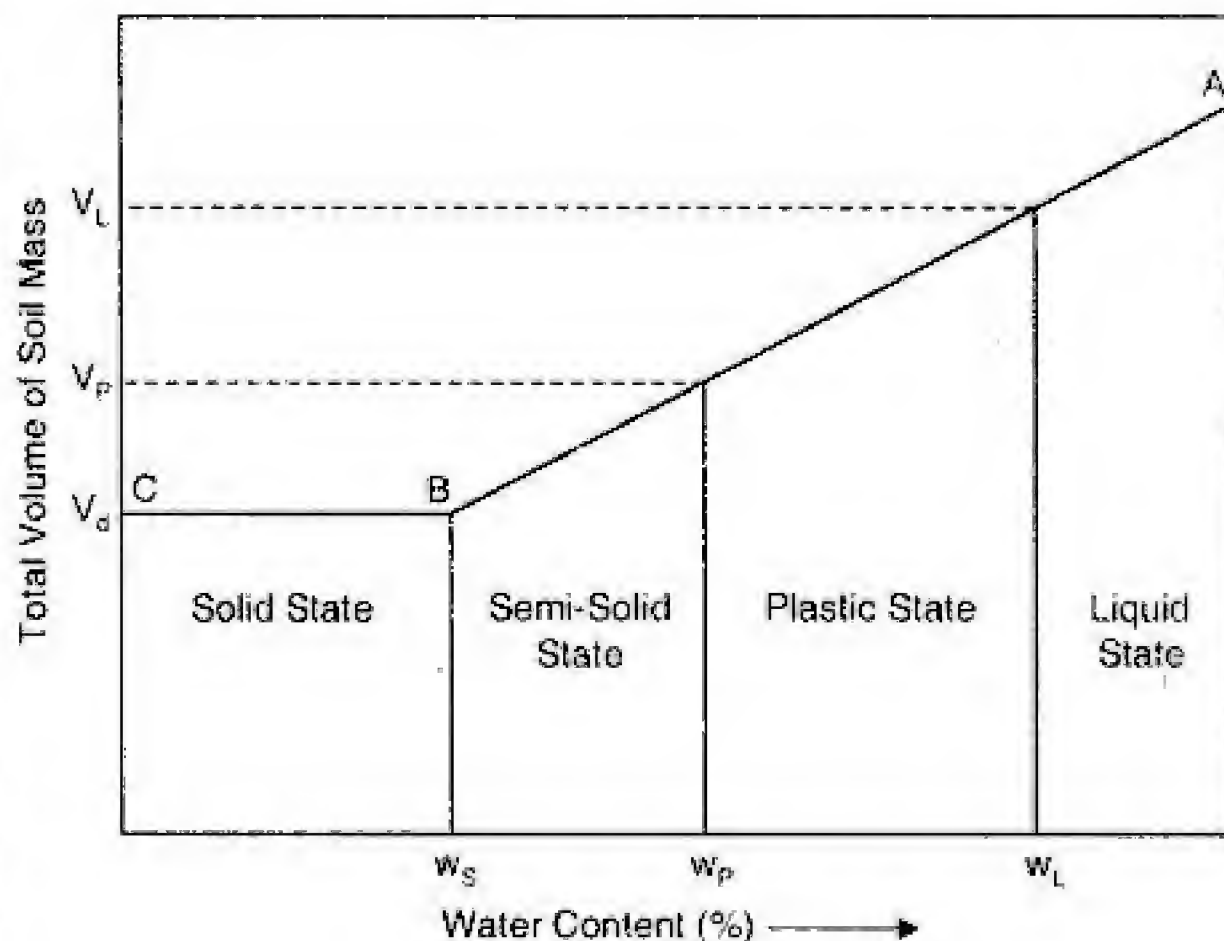


FIG. 3.9. CONSISTENCY LIMITS

**Plastic limit ( $w_p$ ):** Plastic limit is the water content corresponding to an arbitrary limit between the plastic and the semi-solid states of consistency of a soil. It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3 mm in diameter.

**Shrinkage limit ( $w_s$ ).** Shrinkage limit is defined as the maximum water content at which a reduction in water content will not cause a decrease in the volume of a soil mass. It is lowest water content at which a soil can still be completely saturated.

**Plasticity index ( $I_p$ ).** The range of consistency within which a soil exhibits plastic properties is called *plastic range* and is indicated by *plasticity index*. The *plasticity index* is defined as the numerical difference between the liquid limit and the plastic limit of a soil :

$$I_p = w_L - w_P \quad \dots(3.23)$$

In the case of sandy soils, plastic limit should be determined first. When plastic limit cannot be determined, the plasticity index is reported as *NP* (non-plastic). When the plastic limit is equal to or greater than the liquid limit, the plasticity index is reported as zero.

**Plasticity.** Plasticity is defined as that property of a soil which allows it to be deformed rapidly, without rupture, without elastic rebound, and without volume change. According to Goldschmidt theory, the plasticity is due to the presence of thin scale like particles which carry on their surfaces electro-magnetic charges. Water molecules are bi-polar and orient themselves like tiny magnets in the magnetic field next to the surface of the soil particles. Water becomes highly viscous near the particles, but as the distance increases, the viscosity of water decreases until at some distance ordinary water exists. When enough water is present (corresponding to the plastic state of consistency), the particles are separated by molasses-like water which allows particles to slip past each other to new positions without any tendency to return to their former positions, without change in volume of voids, and without impairing the cohesion. The correctness of Goldschmidt's theory for the cause of plasticity is evidenced by the fact that clay does not become plastic when mixed with a liquid of non-polarizing molecules like kerosene.

**Consistency index ( $I_c$ ).** The *consistency index* or the *relative consistency* is defined as the ratio of the liquid limit minus the natural water content to the plasticity index of a soil :

$$I_c = \frac{w_L - w}{I_p} \quad \dots(3.24)$$

where  $w$  is the natural water content of the soil.

Consistency index is useful in the study of the field behaviour of saturated fine grained soils. Thus, if the consistency index of a soil is equal to unity, it is at the plastic limit. Similarly, a soil with  $I_c$  equal to zero is at its liquid limit. If  $I_c$  exceeds unity, the soil is in a semi-solid state and will be stiff. A negative consistency index indicates that the soil has natural water content greater than the liquid limit and hence behaves just like a liquid.

**Liquidity index ( $I_L$ ).** The *liquidity index* or *water-plasticity ratio* is the ratio, expressed as a percentage, of the natural water content of a soil minus its plastic limit, to its plasticity index :

$$I_L = \frac{w - w_P}{I_p} \quad \dots(3.25)$$

where  $w$  is the natural water content of the soil.



### 3.11. DETERMINATION OF LIQUID AND PLASTIC LIMITS

The liquid limit is determined in the laboratory with the help of the standard liquid limit apparatus designed by Casagrande. The apparatus (Fig. 3.10) consists of a hard rubber base of B.S. hardness 21–25, over which a brass cup drops through a desired height. The brass cup can be raised and lowered to fall on the rubber base with the help of a cam operated by a handle. The height of fall of the cup can be adjusted with the help of adjusting screws. Before starting the test, the height of fall of the cup is adjusted to 1 cm. Two types of grooving tools are used : (i) the Casagrande (BS) tool and (ii) ASTM tool. The IS : 9259–1979 designates these tools as *grooving tool (a)* and *grooving tool (b)* respectively. The Casagrande tool cuts a groove of size 2 mm wide at the bottom, 11.0 mm wide at the top and 8 mm high while the ASTM tool cuts a groove 2 mm wide at the bottom, 13.6 mm at the top and 10 mm deep. The ASTM tool is used only for more sandy soils where the Casagrande tool tends to tear the sides of the groove.

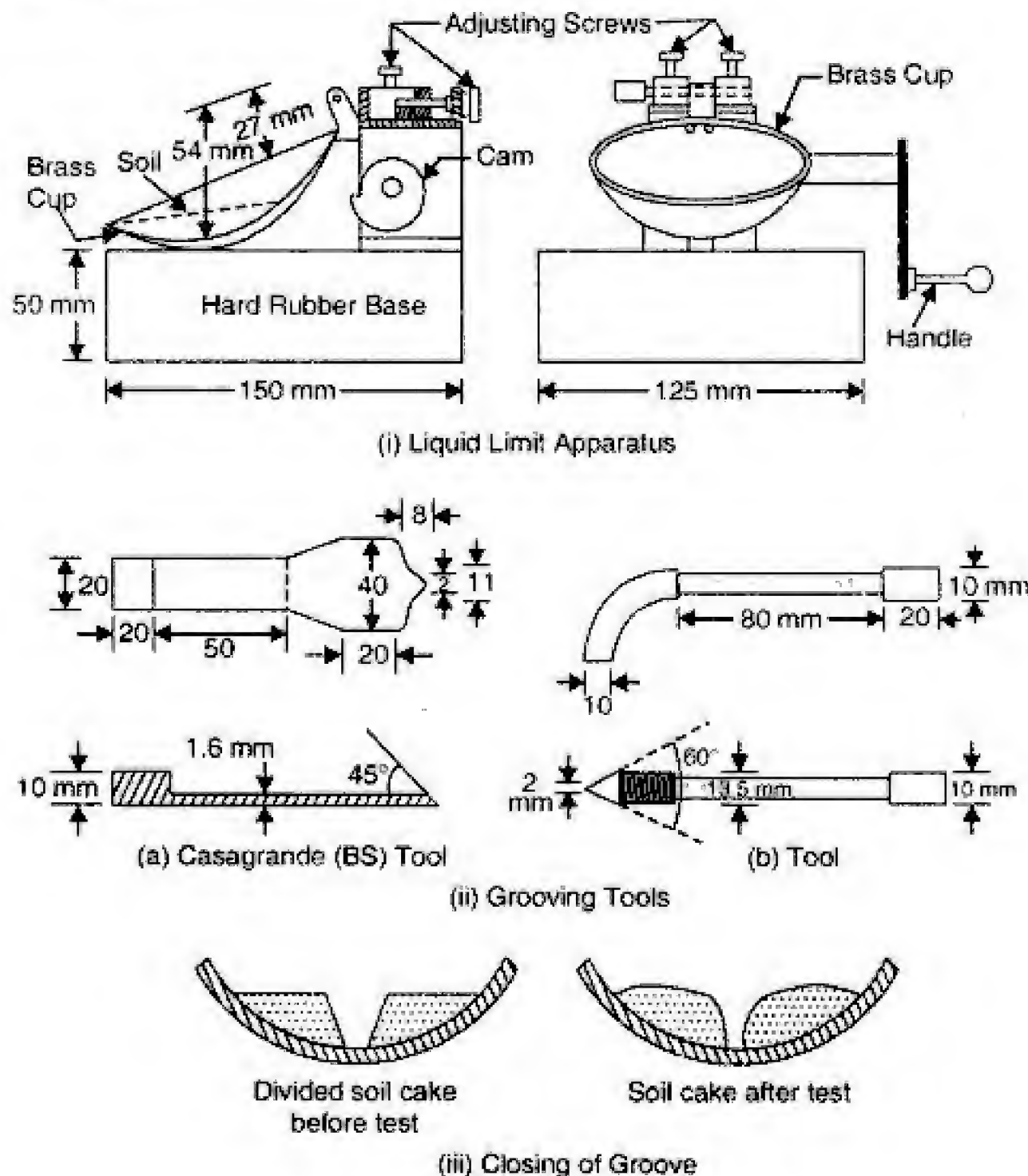


FIG. 3.10. LIQUID LIMIT APPARATUS.

About 120 g of the specimen passing through 425 micron sieve is mixed thoroughly with distilled water in the evaporation dish or on a marble plate to form a uniform paste. A portion of the paste is placed in the cup over the spot where the cup rests on the base, squeezed down and spread into position and the groove is cut in the soil pat [Fig. 3.10 (iii)]. The handle is rotated at a rate about 2 revolutions per second, and the number of blows are counted until the two parts of the soil sample come into contact at the bottom of the groove along a distance of 10 mm. Some soils tend to slide on the surface of the cup instead of the flowing. If this occurs, the result should be discarded and the test repeated until flowing does not occur. After recording the number of blows, approximately 10 gram of soil from near the closed groove is taken for water content determination. Since it is difficult to adjust the water content precisely equal to the liquid limit when the groove should close in 25 blows, the liquid limit is determined by plotting a graph between number of blows as abscissa on a logarithmic scale and the corresponding water content as ordinate. Experience shows that such a graph, known as the *flow curve*, is a straight line having the following equation :

$$w_1 - w_2 = I_f \log_{10} \frac{n_2}{n_1} \quad \dots(3.26)$$

where  $w_1$  = water content corresponding to blows  $n_1$   
 $w_2$  = water content corresponding to blows  $n_2$   
 $I_f$  = slope of the curve, known as the *flow index*.

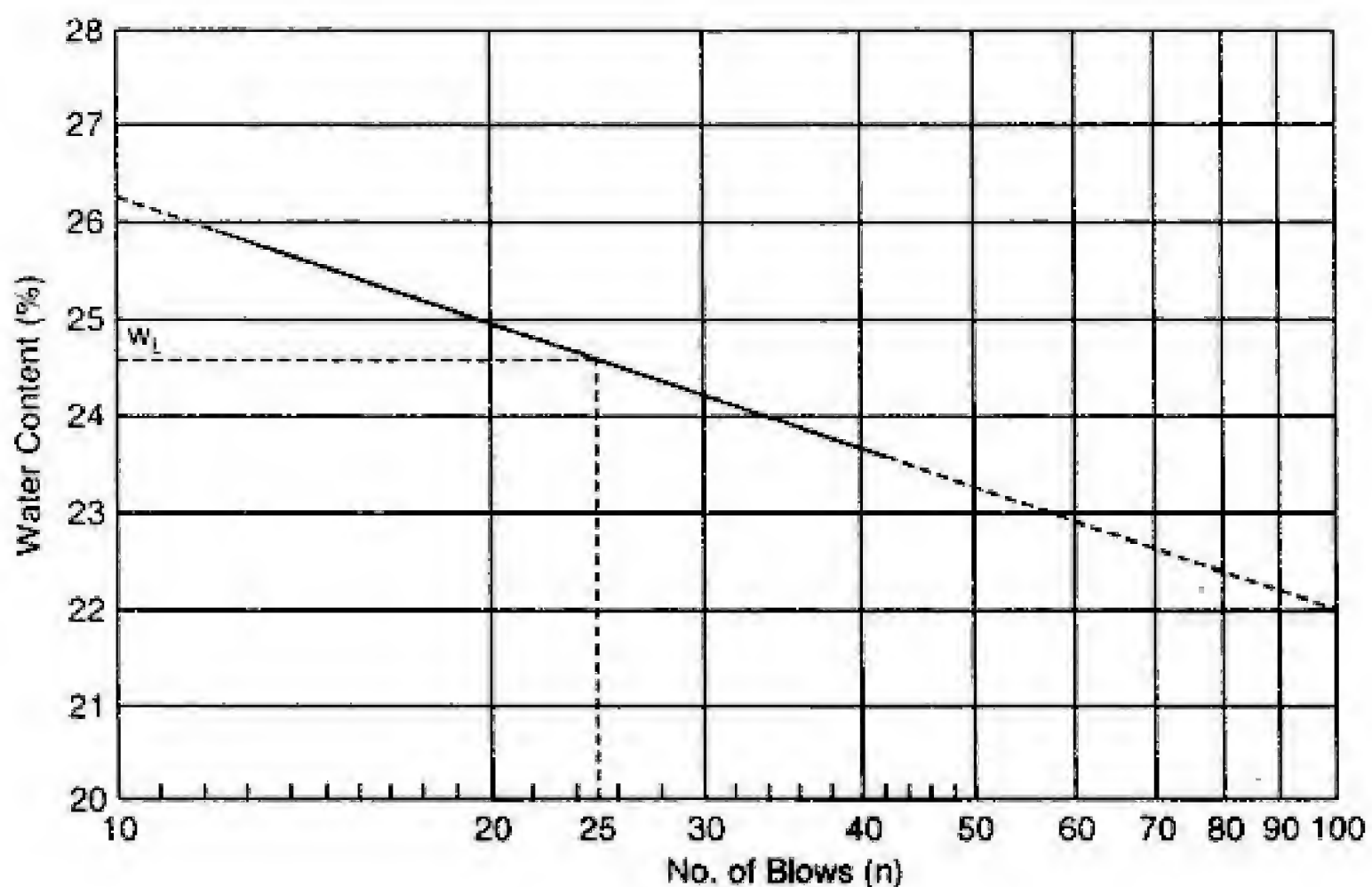


FIG. 3.11. FLOW CURVE.

For plotting the flow curve (Fig. 3.11), at least four to five sets of reading in the range of 10 to 50 blows should be taken. The water content corresponding to 25 blows is taken as the liquid limit.

**Flow index.** The flow index or the slope of the curve can be determined from the relation :

$$I_f = \frac{w_1 - w_2}{\log_{10} \frac{n_2}{n_1}} \quad \dots(3.26 \ a)$$

Selecting the values of  $n_2$  and  $n_1$  corresponding to the number of blows over one log-cycle difference,  $\log_{10} n_2/n_1$  becomes equal to unity, and hence  $I_f$  becomes equal to the difference between the corresponding water contents. Thus, if the flow curve is extended at either end so as to intersect the ordinates corresponding to 10 and 100 blows the numerical difference in water contents at 10 and 100 blows gives directly the flow index.

Table 3.17 gives the observation sheet for the liquid limit test (See Experiment 11).

**One-point method.** Attempts have been made to determine the liquid limit by taking only one reading of water content and its corresponding number of blows. The liquid limit is then estimated from the following equation :

$$w_L = w_n \left( \frac{n}{25} \right)^e \quad \dots(3.27 \ a)$$

where  $w_n$  = water content corresponding to  $n$  number of blows

$w_L$  = water content at liquid limit

$e$  = index, the value of which varies from 0.068 to 0.121 (Mohan : 1959).

If the value of  $n$  is taken between 20 to 30 blows the index  $e$  can be taken to be 0.1, and resulting value of  $w_L$  is fairly accurate. However, this method is used only for getting rough value of the liquid limit.

Alternatively, for the range of blows between 15 and 35, the liquid limit water content ( $w_L$ ) is computed employing the following formula (IS : 2720-1985).

$$w_L = \frac{w_n}{1.3213 - 0.23 \log_{10} n} \quad \dots(3.27)$$

### Static cone penetration method.

The Soviet liquid limit device is based on the principle of static penetration. A  $30^\circ$  cone of stainless steel is made to penetrate the soil pat, under a mass of 75 grams inclusive of the mass of the cone. If the cone penetrates through a depth of 1 cm in 5 seconds, the soil pat is at the liquid limit.

IS : 2720 (Part V) : 1985 specifies a similar penetrometer (Fig. 3.12) for the determination of liquid limit. The cone has a central angle of  $31^\circ$  and total sliding mass of 80 g. The soil pat is kept in a cylindrical trough, 5 cm in diameter and 5 cm high, below the cone. The liquid limit of the soil corresponds to the water content of a paste which would give 20 mm penetration of the soil.



Soil pats are prepared at various water contents and depth of penetration ( $x$ ) for each pat is noted. A graph is plotted representing water content ( $w$ ) on the y-axis and cone penetration ( $x$ ) on the x axis. The best fitting straight line is then drawn. The water content corresponding to a cone penetration of 20 mm is then taken as the liquid limit. The sets of values used for the graphs should be such that the values of penetration are in the range of 14 to 28 mm.

Alternatively,  $w_L$  can be computed by *one-point method* using cone penetrometer, from any one of the following relationships :

$$w_L = \frac{w}{0.77 \log_{10} x} \quad \dots(3.28 \ a)$$

or 
$$w_L = \frac{w}{0.65 + 0.0175 x} \quad \dots(3.28 \ b)$$

The above expression is applicable only if the depth of penetration is between 20 to 30 mm. The expression is based on the assumption that at the liquid limit the shear strength of the soil is about  $1.76 \text{ kN/m}^2$  ( $17.6 \text{ g/cm}^2$ ) which the penetrometer gives for a depth of 25 mm under a total sliding mass of 148 g.

#### PLASTIC LIMIT DETERMINATION.

To determine the plastic limit, the soil specimen, passing 425 micron sieve, is mixed thoroughly with distilled water until the soil mass becomes plastic enough to be easily moulded with fingers. The plastic soil mass should be left for enough time to allow water to permeate through the soil mass. A ball is formed with about 8 g of this plastic soil mass and rolled between the fingers and a glass plate (or marble plate) with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. When a diameter of 3 mm is reached, the soil is remoulded again into a ball. This process of rolling and remoulding is repeated until the thread starts just crumbling at a diameter of 3 mm. The crumbled threads are kept for water content determination. The test is repeated twice more with fresh samples. The plastic limit  $w_p$  is then taken as the average of three water contents.

The *plasticity index* is calculated from the relation :

$$I_p = w_L - w_p$$

**Toughness Index ( $I_T$ ).** The toughness index is defined as the ratio of the plasticity index to the flow index :

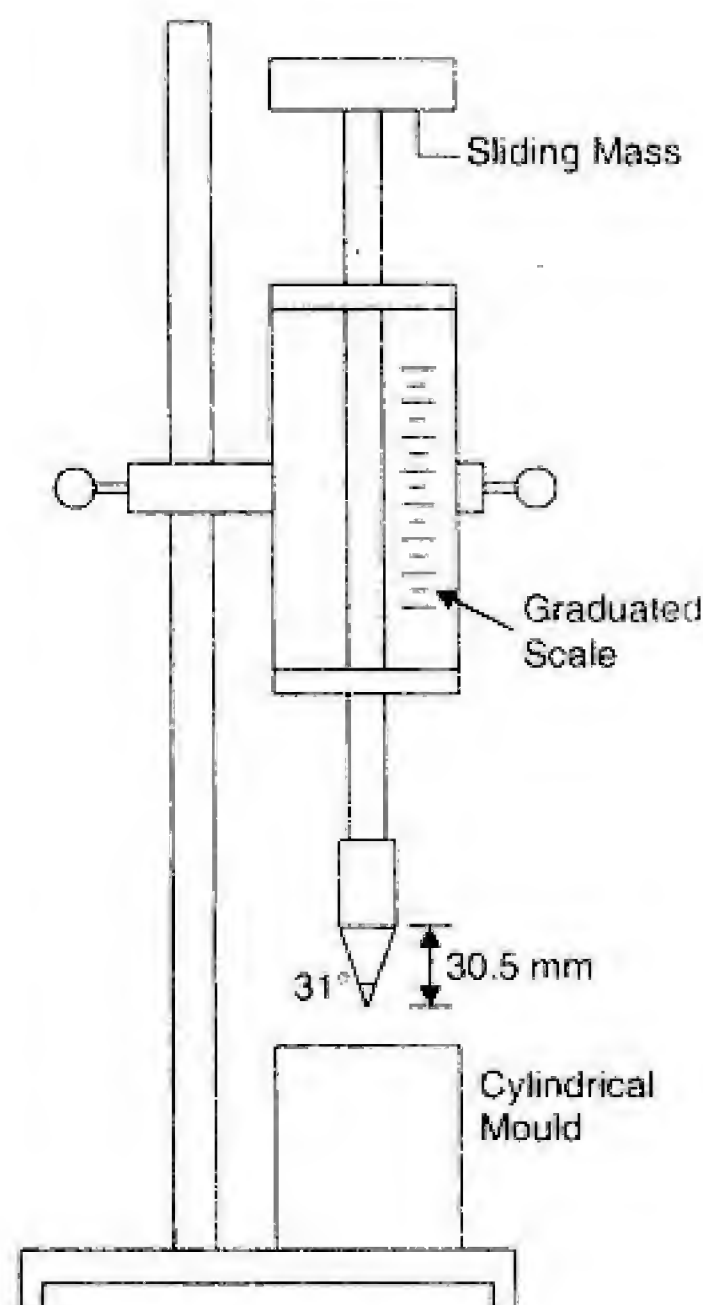


FIG. 3.12. CONE PENETRATION.  
IS : 2720 (PART V)-1965.

$$I_T = I_P / I_f \quad \dots(3.29)$$

Knowing  $I_P$  and  $I_f$ ,  $I_T$  can be computed. Similarly, if the natural water content of a soil is known, its consistency index, and liquidity index can be calculated using Eqs. 3.24 and 3.25.

### 3.12 SHRINKAGE LIMIT

An expression for the shrinkage limit of a soil can be derived with reference to Fig. 3.13. If a saturated soil sample

is taken (having water content more than the shrinkage limit) and allowed to dry up gradually, its volume will go on reducing till a stage will come after which the reduction in the soil water will not result in further reduction in the total volume of the soil sample. The water content corresponding to that stage is known as the shrinkage limit. Fig. 3.13 (a) shows such a soil sample of volume  $V_1$  and mass  $M_1$  while Fig. 3.13 (b)

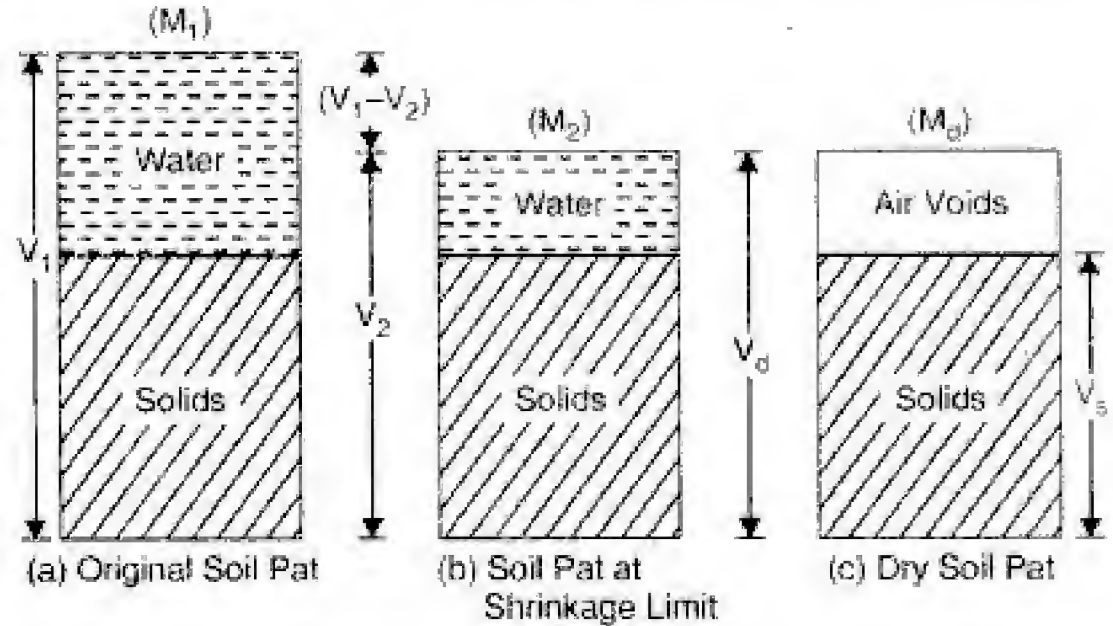


FIG. 3.13. SHRINKAGE LIMIT DETERMINATION.

is the stage of the sample at its

shrinkage limit. Fig. 3.13 (c) is the oven-dried sample having volume  $V_d$  and mass  $M_d$ . Evidently, according to the definition, volumes  $V_2$  and  $V_d$  are equal.

Now, mass of water in (a) =  $M_1 - M_d$

Loss of water from (a) to (b) =  $(V_1 - V_2) \rho_w$

$\therefore$  Mass of water in (b) =  $(M_1 - M_d) - (V_1 - V_2) \rho_w$

$$\therefore w_s = \frac{(M_1 - M_d) - (V_1 - V_2) \rho_w}{M_d} \times 100 \quad \dots(3.30 \ a)$$

$$\text{or } w_s = \left[ w_1 - \frac{(V_1 - V_d) \rho_w}{M_d} \right] \times 100 \quad \dots(3.30)$$

$$\text{or } w_s = \left[ w_1 - \frac{(V_1 - V_d) \gamma_w}{W_d} \right] \times 100 \quad \dots(3.30 \ b)$$

where  $w_1$  = water content of the original saturated sample of volume  $V_1$

$V_d$  = dry volume of the soil sample ;  $W_d$  = dry weight of the soil sample

$M_d$  = dry mass of the soil sample.

The form of Eq. 3.30 immediately suggests the method of determining the shrinkage limit in the laboratory. The equipment for the determination consists of (i) a porcelain evaporating dish, about 12 cm in diameter with flat bottom, (ii) a stainless steel shrinkage dish, 45 mm in diameter and 15 mm in height, with flat bottom, (iii) two glass plates, each 75 × 75 mm, one of plain glass and the other having three metal prongs, and (iv)

a glass cup 50 mm in diameter and 25 mm in height, with its top rim ground smooth and level (Fig. 3.14).

The volume  $V_i$  of the shrinkage dish is first determined by filling it to overflow with mercury, removing the excess by pressing a flat glass plate over its top and then taking the mass of the dish filled with mercury. The mass of the mercury contained in the dish, divided by its density ( $13.6 \text{ g/cm}^3$ ) gives the volume of the dish. About 50 g of soil passing 425-micron IS sieve is mixed with distilled water sufficient to fill the voids completely and to make the soil pasty enough to be readily worked into the shrinkage dish without the inclusion of air-bubbles. The inside of the shrinkage dish is coated with a thin layer of vaseline. A volume of wet soil of about one-third the volume of dish is put in its centre and the soil is caused to flow to the edges by tapping it gently on a hard surface. The dish is gradually filled by adding more soil in instalments followed by gently tappings to exclude the inclusion of air. The excess soil is struck off with straight edge and all soil adhering to the outside of the dish is wiped off. The dish filled with soil is then immediately weighed. The mass  $M_i$  of the wet soil pat, of volume  $V_i$ , is thus known by subtracting the mass of the empty dish from the mass of the wet soil plus the dish taken above. The dish is then placed in the oven. The soil pat will have volumetric shrinkage on drying, as shown in Fig. 3.14 (b). The mass  $M_d$  of the dry soil pat is found. To find the volume  $V_d$  of the dry soil pat, the glass cup is first filled with mercury and the excess mercury is removed by pressing the glass plate with three prongs firmly over the top of the cup. The cup is wiped off any mercury which may be adhering to its outside surface, and is placed in the evaporating dish. The dry soil pat is placed on the surface of the mercury of the cup and is carefully forced down by means of glass with prongs. The mass of the mercury so displaced divided by its density gives the volume  $V_d$  of the dry soil pat. The shrinkage limit is then calculated from Eq. 3.30. Table 3.20 gives the observation sheet for the shrinkage limit determination (See Experiment 13).

The dish is gradually filled by adding more soil in instalments followed by gently tappings to exclude the inclusion of air. The excess soil is struck off with straight edge and all soil adhering to the outside of the dish is wiped off. The dish filled with soil is then immediately weighed. The mass  $M_i$  of the wet soil pat, of volume  $V_i$ , is thus known by subtracting the mass of the empty dish from the mass of the wet soil plus the dish taken above. The dish is then placed in the oven. The soil pat will have volumetric shrinkage on drying, as shown in Fig. 3.14 (b). The mass  $M_d$  of the dry soil pat is found. To find the volume  $V_d$  of the dry soil pat, the glass cup is first filled with mercury and the excess mercury is removed by pressing the glass plate with three prongs firmly over the top of the cup. The cup is wiped off any mercury which may be adhering to its outside surface, and is placed in the evaporating dish. The dry soil pat is placed on the surface of the mercury of the cup and is carefully forced down by means of glass with prongs. The mass of the mercury so displaced divided by its density gives the volume  $V_d$  of the dry soil pat. The shrinkage limit is then calculated from Eq. 3.30. Table 3.20 gives the observation sheet for the shrinkage limit determination (See Experiment 13).

**Alternative method.** Alternatively shrinkage limit can be found if the specific gravity  $G$  of the soil grains is known. If  $V_s$  is the volume of solids, and  $V_d$  is the dry volume, we have

$$w_s = \frac{(V_d - V_s) \rho_w}{M_d} \times 100 = \left( V_d - \frac{M_d}{\rho_s} \right) \frac{\rho_w}{M_d} \times 100 \quad \dots(3.31)$$

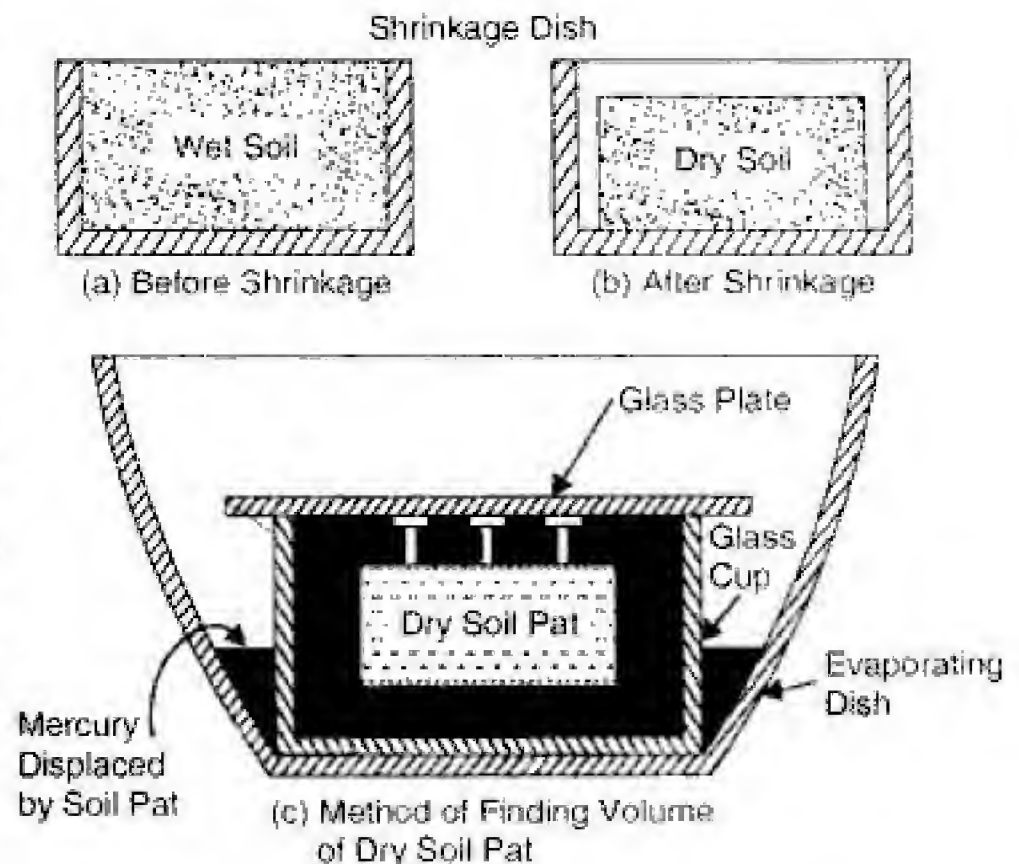


FIG. 3.14. APPARATUS OF SHRINKAGE LIMIT DETERMINATION.



where

$\rho_s$  = density of soil solids

$$\therefore w_s = \left( \frac{V_d}{M_d} \rho_w - \frac{1}{G} \right) 100 = \left( \frac{V_d}{M_d} - \frac{1}{G} \right) 100 \quad \dots(3.31 \ a)$$

(Since  $\rho_w = 1 \text{ (g/cm}^3\text{)}$ )

Eq. 3.31 may also be written as

$$w_s = \left( \frac{\rho_w}{\rho_d} - \frac{1}{G} \right) 100 = \left( \frac{\gamma_w}{\gamma_d} - \frac{1}{G} \right) 100 \quad \dots(3.31 \ b)$$

where

$\rho_d$  = dry density of soil based on its minimum (dry) volume

$\gamma_d$  = dry unit weight of soil sample.

Substituting  $\gamma_d = \frac{G \gamma_w}{1 + e}$  in Eq. 3.31 (b), we get

$$w_s = \frac{e}{G} \quad \dots(3.32)$$

where  $e$  = voids ratio of the sample at its minimum (dry) volume.

**Approximate value of  $G$  from shrinkage limit test :** The observation of a shrinkage limit test can also be used to determine the approximate value of  $G$  as under :

$$\gamma_s = G \gamma_w = \frac{M_d \times 9.81}{V_s} \quad \text{Hence } G = \frac{M_d}{V_s} \quad (\text{Since } \gamma_w = 9.81 \text{ kN/m}^3)$$

$$\text{But } V_s = V_1 - \frac{M_1 - M_d}{\rho_w} \quad [\text{From Fig. 3.13 (a)}]$$

$$\therefore G = \frac{M_d \rho_w}{V_1 \rho_w - (M_1 - M_d)} \quad (\text{where } \rho_w = 1 \text{ g/cm}^3) \quad \dots(3.33 \ a)$$

$$\therefore G = \frac{M_d}{V_1 - (M_1 - M_d)} \quad \dots(3.33)$$

Also, if the shrinkage limit is known, we get from Eq. 3.31 (b)

$$G = \frac{1}{\frac{\gamma_w}{\gamma_d} - \frac{w_s}{100}} = \frac{1}{\frac{\rho_w}{\rho_d} - \frac{w_s}{100}} \quad \dots(3.34)$$

where  $\rho_d$  = dry density of soil ( $\text{g/cm}^3$ ) ;  $\rho_w$  = density of water ( $= 1 \text{ g/cm}^3$ ).

**Shrinkage ratio (SR).** We have seen that if a soil slurry is allowed to dry up, the reduction in water content of the saturated soil mass is accompanied by a change in the volume of the soil mass, the total change in the volume of the soil mass is equal to the volume of water evaporated. This reduction in volume takes place upto the shrinkage limit. The reduction in the volume is represented by the straight line  $AB$  in Fig. 3.9. *Shrinkage ratio* is defined as the ratio of a given volume change expressed as a percentage of dry volume, to the corresponding change in water content above the shrinkage limit expressed as a percentage of the weight of the oven dried soil :

$$SR = \frac{\frac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2} \quad \dots(3.35)$$

where  $V_1$  = volume of soil mass at water content  $w_1$   
 $V_2$  = volume of soil mass at water content  $w_2$   
 $V_d$  = volume of dry soil mass

$w_1, w_2$  = water content, expressed as percentage.

At the shrinkage limit [Fig. 3.13 (b)],  $V_2 = V_d$  and  $w_2 = w_s$  ; hence

$$SR = \frac{\left( \frac{V_1 - V_d}{V_d} \right) 100}{w_1 - w_s} \quad \dots(3.36)$$

Also, in Eq. 3.35, the change in the water contents ( $w_1 - w_2$ ) is given by

$$w_1 - w_2 = \frac{(V_1 - V_2) \rho_w}{M_d} \times 100$$

$$\text{Hence} \quad SR = \frac{M_d}{V_d \rho_w} = \frac{\rho_d}{\rho_w} = \frac{\gamma_d}{\gamma_w} \quad \dots(3.37)$$

*Thus, the shrinkage ratio of a soil is equal to the mass specific gravity of the soil in its dry state.* The shrinkage limit test data can be substituted in Eq. 3.36 to determine the shrinkage ratio of the soil.

**Volumetric shrinkage (VS).** The volumetric shrinkage or *volumetric change* is defined as the decrease in the volume of a soil mass, expressed as a percentage of the dry volume of the soil mass, when the water content is reduced from a given percentage to the shrinkage limit :

$$VS = \frac{V_1 - V_d}{V_d} \times 100 \quad \dots(3.38 \ a)$$

$$\text{But } \frac{V_1 - V_d}{V_d} \times 100 = (w_1 - w_s) SR \quad \text{Eq.(3.36)}$$

$$\therefore VS = (w_1 - w_s) SR \quad \dots(3.38)$$

where  $V_1$  is the volume of soil mass at any water content  $w_1$  .

**Linear shrinkage ( $L_s$ ).** It is defined as the decrease in one dimension of a soil mass expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit. It is calculated from the following formula:

$$L_s = 100 \left[ 1 - \left( \frac{100}{VS + 100} \right)^{\frac{1}{3}} \right] \quad \dots(3.39)$$

**Example 3.6.** An undisturbed saturated specimen of clay has a volume of  $18.9 \text{ cm}^3$  and a mass of  $30.2 \text{ g}$ . On oven drying, the mass reduces to  $18.0 \text{ g}$ . The volume of dry specimen as determined by displacement of mercury is  $9.9 \text{ cm}^3$ . Determine shrinkage limit, specific gravity, shrinkage ratio and volumetric shrinkage.

**Solution :** Given :  $M_1 = 30.2 \text{ g}$  ;  $M_d = 18.0 \text{ g}$  ;  $\rho_w = 1 \text{ g/cm}^3$

$$V_1 = 18.9 \text{ cm}^3 ; V_2 = 9.9 \text{ cm}^3$$

$$(i) \quad w_s = \left[ \frac{M_1 - M_d}{M_d} - \frac{(V_1 - V_2) \rho_w}{M_d} \right] \times 100 \quad \dots(3.30 \ a)$$

$$= \left[ \frac{30.2 - 18.0}{18.0} - \frac{(18.9 - 9.9) 1}{18.0} \right] \times 100 = 17.8 \%$$

$$(ii) \quad G = \frac{M_d}{V_1 - (M_1 - M_d)} = \frac{18.0}{18.9 - (30.2 - 18.0)} = 2.69. \quad \dots(3.33)$$

Alternatively, from Eq. 3.34 ;  $G = \frac{1}{\left( \frac{\rho_w}{\rho_d} - \frac{w_s}{100} \right)}$

where  $\rho_w$  = density of water = 1 g/cm<sup>3</sup>

$$\rho_d = \text{dry density of soil specimen} = \frac{18.0}{9.9} = 1.818 \text{ g/cm}^3$$

$$\therefore G = \frac{1}{\left( \frac{1}{1.818} - \frac{17.8}{100} \right)} = 2.69.$$

$$(iii) \text{ Shrinkage ratio} \quad SR = \frac{\gamma_d}{\gamma_w} = \frac{\rho_d}{\rho_w} = \frac{1.818}{1} \approx 1.82.$$

$$(iv) \text{ Volumetric shrinkage} \quad VS = \frac{(V_1 - V_d) 100}{V_d} = \frac{18.9 - 9.9}{9.9} \times 100 = 91 \%$$

$$\text{Alternatively,} \quad VS = (w_1 - w_s) SR = (67.8 - 17.8) \times 1.82 = 91 \%$$

**Example 3.7.** The mass specific gravity of a fully saturated specimen of clay having a water content of 36% is 1.86. On oven drying, the mass specific gravity drops to 1.72. Calculate the specific gravity of clay and its shrinkage limit.

**Solution:**  $e = w_{sat} \cdot G = 0.36 G$

$$\text{Mass specific gravity, } G_m = \left[ \frac{(G + e) \gamma_w}{1 + e} \right] \frac{1}{\gamma_w}$$

$$\therefore 1.86 = \frac{G + 0.36 G}{1 + 0.36 G} = \frac{1.36 G}{1 + 0.36 G}$$

$$\text{From which,} \quad G = 2.69$$

$$\text{Now} \quad w_s = \frac{\gamma_w}{\gamma_d} - \frac{1}{G} \quad \text{where} \quad \frac{\gamma_d}{\gamma_w} = \text{mass specific gravity of dry soil} = 1.72$$

$$\therefore w_s = \frac{1}{1.72} - \frac{1}{2.69} = 0.21 = 21 \%$$

$$\text{Alternatively, } (e)_{dry} = \frac{G \gamma_w}{\gamma_d} - 1 = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.69 \times 1}{1.72} - 1 = 0.565$$

$$w_s = \frac{e}{G} = \frac{0.565}{2.69} = 0.21 = 21 \%$$

**Example 3.8.** The Atterberg limits of a clay soil are : liquid limit 52%, plastic limit 30% and shrinkage limit 18%. If the specimen of this soil shrinks from a volume



of  $39.5 \text{ cm}^3$  at the liquid limit to a volume of  $24.2 \text{ cm}^3$  at the shrinkage limit, calculate the true specific gravity.

**Solution :** Fig. 3.15 (a, b, c) shows that the states of the specimen at liquid limit, shrinkage limit and dry condition respectively.

Difference of volume of water in (a) and (b) =  $39.5 - 24.2 = 15.3 \text{ cm}^3$

Difference of mass of water in (a) and (b) =  $15.3 \text{ g}$ .

But from Fig. 3.15 (a), (b), this difference is equal to  $(0.52 - 0.18) M_d$

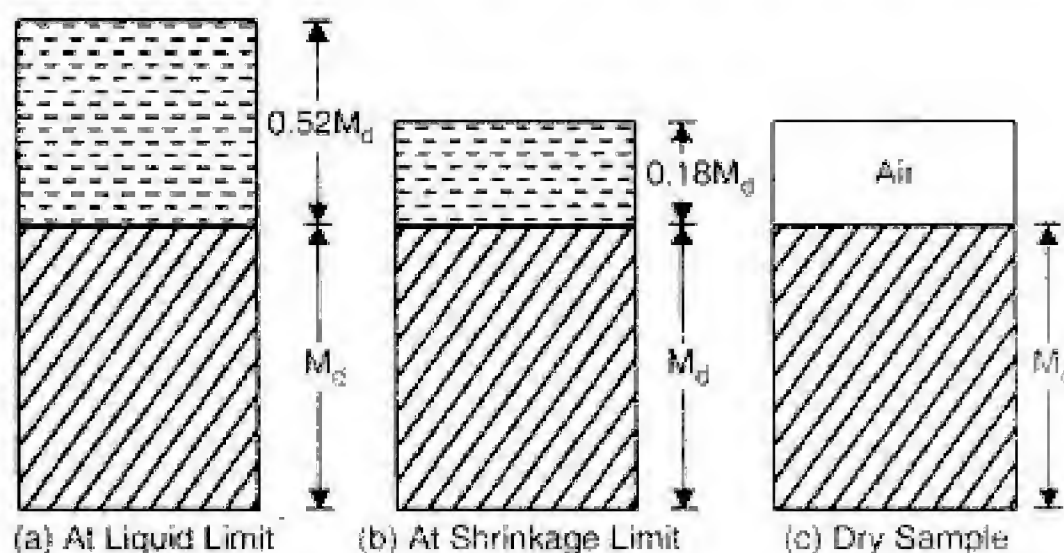


FIG. 3.15

$$\therefore (0.52 - 0.18) M_d = 15.3 \quad \text{or} \quad M_d = \frac{15.3}{0.34} = 45 \text{ g}$$

$$\text{Mass of water in (b)} = 0.18 M_d = 0.18 \times 45 = 8.1 \text{ g}$$

$$\text{Volume of water in (b)} = 8.1 \text{ cm}^3$$

$$\therefore \text{Volume of solids, } V_s \text{ in (b)} = 24.2 - 8.1 = 16.1 \text{ cm}^3$$

$$\text{Hence } \rho_s = \text{density of solids} = \frac{M_d}{V_s} = \frac{45}{16.1} = 2.8 \text{ g/cm}^3$$

$$\therefore G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w} = \frac{2.8}{1} = 2.8.$$

**Example 3.9.** The plastic limit of a soil is 25% and its plasticity index is 8%. When the soil is dried from its state at plastic limit, the volume change is 25% of its volume at plastic limit. Similarly, the corresponding volume change from the liquid limit to the dry state is 34% of its volume at liquid limit. Determine the shrinkage limit and the shrinkage ratio.

**Soltion :**

$$w_p = 25 \% ; I_p = 8 \%$$

$$\therefore w_L = 25 + 8 = 33 \%$$

Volume change at liquid limit = 34%

$\therefore$  Dry volume

$$V_d = V_L - 0.34 V_L = 0.66 V_L$$

where  $V_L$  = volume at liquid limit

...(i)

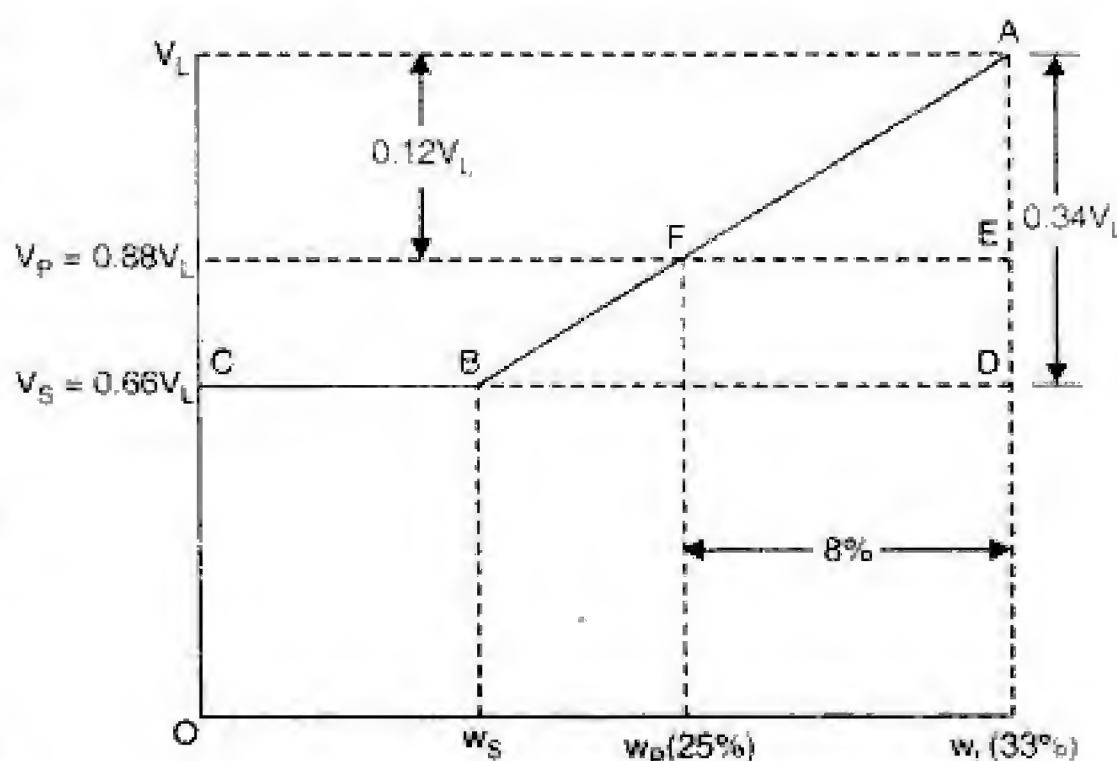


FIG. 3.16

Similarly, volume change at plastic limit = 25%

$$\therefore V_d = V_p - 0.25 V_p = 0.75 V_p \quad \dots(ii)$$

where  $V_p$  = volume at plastic limit

$$\text{Equating (i) and (ii), we get } V_p = \frac{0.66}{0.75} V_L = 0.88 V_L$$

Fig. 3.16 shows the consistency limits. From the diagram, it is clear that when the soil passes from liquid limit to plastic limit, there is a change of  $(1 - 0.88) V_L$  in volume and 8% change in water content.

$$\therefore \frac{BD}{AD} = \frac{FE}{AE} = \frac{8}{0.12 V_L} ; \quad \therefore BD = \frac{8}{0.12 V_L} \times 0.34 V_L = 22.6 \%$$

$$\therefore w_s = w_L - 22.6 \% = 33 - 22.6 = 10.4 \%$$

$$SR = \frac{\frac{V_1 - V_2}{V}}{w_1 - w_2} = \frac{(V_L - V_p) \times 100}{V_d (w_L - w_p)} = \frac{V_L (1 - 0.88) 100}{0.66 V_L (33 - 25)} = \frac{0.12 \times 100}{0.66 \times 8} = 2.27.$$

**Example 3.10.** A saturated soil sample has a volume of  $25 \text{ cm}^3$  at the liquid limit. If the soil has liquid limit and shrinkage limit of 42% and 20% respectively, determine the minimum volume which can be attained by the soil specimen. Take  $G = 2.72$ .

**Solution :** The soil specimen will attain minimum volume at shrinkage limit. Fig. 3.17 (a) and (b) show the states of the specimen at liquid limit and shrinkage limit respectively.

If  $M_d$  is the mass of solids in g, volume of water at liquid limit is

$$V_L = 0.42 M_d \text{ cm}^3$$

Volume of solids,

$$V_s = \frac{M_d}{G \rho_w} = \frac{M_d}{2.72 \times 1} = \frac{M_d}{2.72} \text{ cm}^3 \\ = 0.368 M_d \text{ cm}^3$$

$$\therefore \text{Total volume}$$

$$= 0.42 M_d + 0.368 M_d. \text{ But this is equal to } 25 \text{ cm}^3$$

$$\therefore 0.42 M_d + 0.368 M_d = 25, \text{ from which } M_d = 31.74 \text{ g}$$

At the shrinkage limit, soil attains its minimum volume  $V_m$

$$\therefore V_m = V_s + 0.2 M_d = 0.368 M_d + 0.2 M_d = 0.568 \times 31.74 = 18.03 \text{ cm}^3$$

**Example 3.11** An oven-dried sample of soil has a volume of  $265 \text{ cm}^3$  and a mass of 456 g. Taking  $G = 2.71$ , determine the voids ratio and shrinkage limit. What will be the water content which will fully saturate the soil sample and also cause an increase in volume equal to 10% of the original dry volume ?

$$\text{Solution : Dry density } \rho_d = \frac{M_d}{V} = \frac{456}{265} = 1.721 \text{ g/cm}^3$$

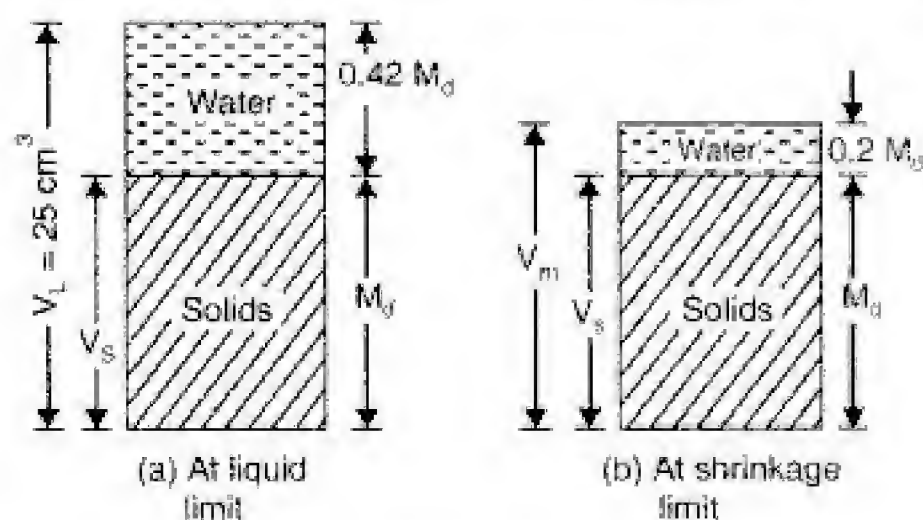


FIG. 3.17

But  $\rho_d = \frac{G \rho_w}{1 + e}$

$\therefore 1.721 = \frac{2.7 \times 1}{1 + e}$  From which

$$e = \frac{2.71}{1.721} - 1 = 0.575$$

Shrinkage limit

$$= \frac{e}{G} = \frac{0.575}{2.71} = 0.212 = 21.2\%$$

Fig. 3.18 (a) and (b) show the states of the specimen at shrinkage limit and desired final state respectively.

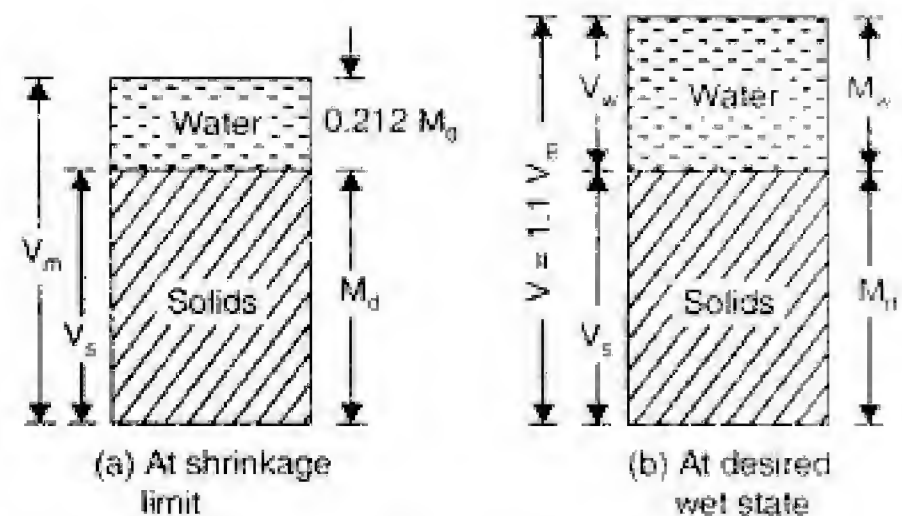


FIG. 3.18

Now,  $V_w = 265 \text{ cm}^3$  (given). Also,  $V_s = \frac{M_d}{G} = \frac{456}{2.71} = 168.27 \text{ cm}^3$

At the final desired state  $V = 1.1 V_m = 1.1 \times 265 = 291.5 \text{ cm}^3$  ... (1)

But from Fig. 3.18 (a),  $V = V_w + V_s = V_w + 168.27$  ... (2)

From (1) and (2), we get  $V_w = 291.5 - 168.27 = 123.23 \text{ cm}^3$

$\therefore$  Mass of water in final state,  $M_w = 123.23 \text{ g}$

Hence water content in final state  $= \frac{M_w}{M_d} = \frac{123.23}{456} = 0.27 = 27\%$

### 3.13. DETERMINATION OF IN-SITU DENSITY, VOIDS RATIO AND DENSITY INDEX

The field density of a natural soil deposit or of a compacted soil can be determined by the following methods :

1. Sand replacement method
2. Core cutter method
3. Water displacement method
4. Rubber balloon method.

**1. Sand Replacement method.** The equipment in the sand replacement method consists of (i) sand pouring cylinder mounted above a pouring cone and separated by a valve or shutter, (ii) calibrating container, (iii) tray with central circular hole, and (iv) chisel, scoop, balance etc. The procedure consists of (a) calibration of the cylinder, (b) measurement of a soil density, and (c) determination of water content and dry density.

**(a) Calibration of the cylinder.** This consists of the determination of the weight of sand required to fill the pouring cone of the cylinder, and the determination of the bulk density of sand. Uniformly graded, dry, clean sand preferably passing a 600 micron sieve and retained on 300-micron IS sieve is used in the cylinder. The cylinder is filled upto a height

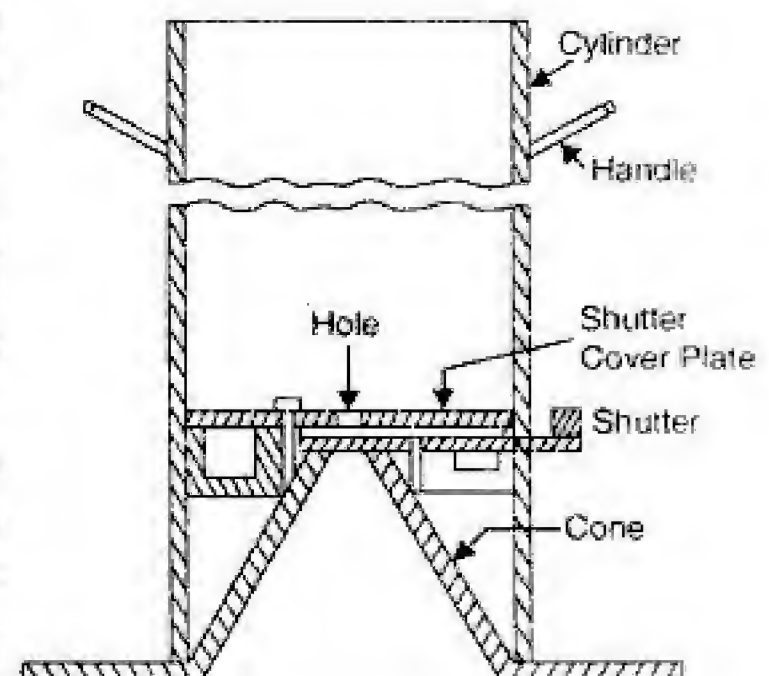


FIG. 3.19. SAND REPLACEMENT CYLINDER.



1 cm below the top, and its initial mass  $M_1$  is taken. The sand is run out of cylinder, equal in volume to that of the calibrating container. The cylinder is then placed over a plane surface and the sand is allowed to run out to fill the cone below.

When no further sand runs out, the valve is closed. The sand filled in the cone is collected, and its mass  $M_2$  is found. All the sand is then refilled in the cylinder so that the total mass of sand and cylinder is equal to the original mass  $M_1$ . The cylinder is then put centrally above the calibrating container, and the sand is allowed to run into the calibrating container. The valve is closed when there is no further movement of sand. The mass of cylinder with sand is found ( $M_3$ ). The mass  $M'$  of the sand required to fill the calibrating container will be equal to  $M_1 - M_3 - M_2$ . The mass  $M'$  divided by the volume of the calibrating container gives the bulk density of the sand. All the sand is then refilled in the cylinder.

(b) **Measurement of soil density.** The site is cleaned and levelled, and the tray placed over it. A test hole, approximately of a depth equal to that of the calibrating container is excavated in the ground, and the soil is collected in the tray. The mass  $M$  of the excavated soil is found. The cylinder is centrally placed over the hole, and the sand is allowed to run in it. The valve is closed when no further movement of sand takes place. The mass  $M_4$  of the cylinder and the remaining sand in it is measured. The mass  $M''$  run into the hole, upto level ground surface, will evidently be equal to  $M_1 - M_2 - M_4$ . Dividing  $M''$  by the bulk density of sand, the volume of the hole and hence the volume  $V$  of the excavated soil is known. Dividing the mass  $M$  by the volume  $V$ , the bulk density  $\rho$  of the soil excavated is known.

(c) **Dry density.** A suitable sample of the excavated soil is kept for water content ( $w$ ) determination. The dry density of the soil will be equal to the bulk density divided by  $(1 + w)$ . Table 3.10 gives the observation sheet for the test. The bulk unit weight  $\gamma = \rho g = 9.81 \rho$  ( $\text{kN/m}^3$ ) and the dry unit weight  $\gamma_d = \rho_d \cdot g = 9.81 \rho_d$  ( $\text{kN/m}^3$ ). [See Experiment 7]

**2. The core cutter method.** A core cutter, consisting of a steel cutter, 10 cm in diameter and about 13 cm high, and a 2.5 cm high dolly is driven in the cleaned surface with the help of a suitable rammer, till about 1 cm of the dolly protrudes above the surface. The cutter, containing the soil, is dug out of the ground, the dolly is removed and the excess soil is trimmed off. The mass of the soil in the cutter is found. By dividing it by the volume of the cutter the bulk density is determined. The

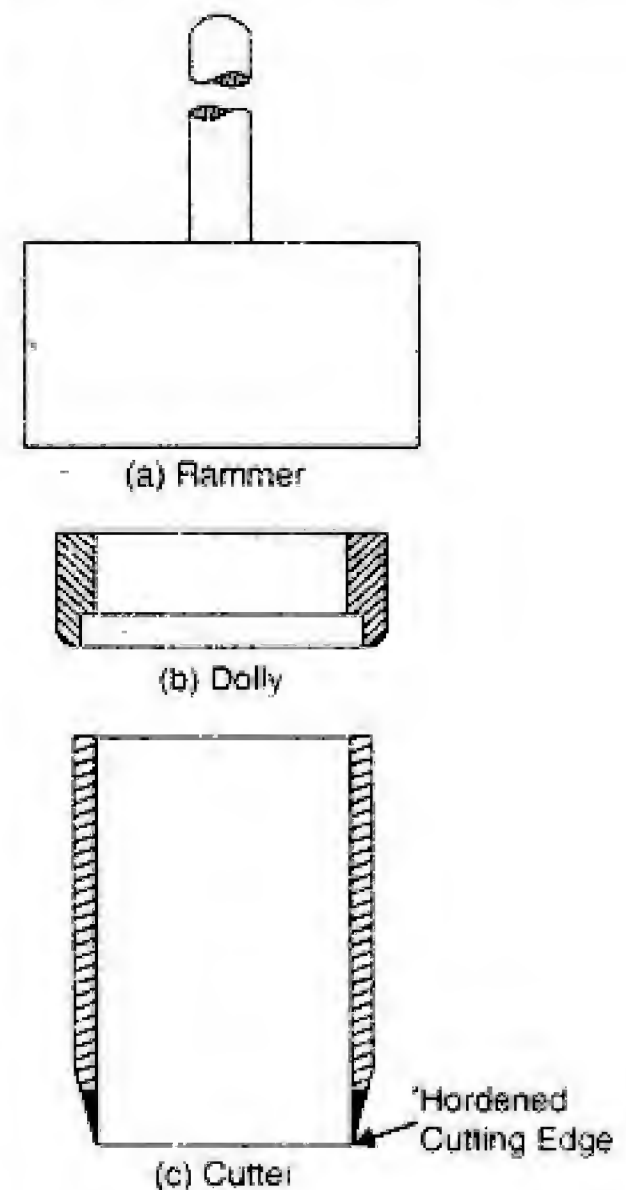


FIG. 3.20. CORE CUTTER.

water content of the excavated soil is found in the laboratory, and the dry density is computed, [See Experiment 6].

**3. Water displacement method.** The method is suitable only for cohesive soil samples brought from the field. A small specimen is trimmed to a more or less regular shape, from a larger sample, and its mass  $M_1$  is found. The specimen is covered with a thin layer of paraffin wax and the mass  $M_2$  of the coated specimen is taken. A metal container is filled above the overflow level, and excess water is allowed to run off through the overflow outlet. The coated specimen is then slowly immersed in the container, and the overflow water is collected in a measuring jar. The volume  $V_w$  of the displaced water is thus known. The volume  $V$  of the uncoated specimen is then calculated from the relation:

$$V = V_w - \frac{M_2 - M_1}{G_p} \quad \dots(3.40)$$

where  $G_p$  = density of paraffin wax (g/ml)

In the absence of any other test,  $G_p$  may be taken as 0.908 g/ml.

The bulk density and the dry density of the specimen are determined from the relations

$$\rho = \frac{M_1}{V} \quad \text{and} \quad \rho_d = \frac{\rho}{1 + w}$$

The bulk unit weight,  $\gamma = \rho \cdot g = \frac{M_1}{V} \times 9.81 \text{ (kN/m}^3\text{)}$  and dry unit weight,  $\gamma_d = \frac{\gamma}{1 + w}$

where  $w$  is the water content of the specimen, to be determined by oven-drying a small specimen. [See Experiment 5].

**4. Submerged mass density method.** This method is used to determine the volume ( $V_w$ ) of the wax coated specimen, and is based on Archimedes' principle that when a body is submerged in water, the reduction in its mass is equal to mass of volume of water displaced. The soil sample is trimmed and its mass  $M_1$  is found. It is then covered with a thin layer of paraffin wax and the mass  $M_2$  of wax coated specimen is found. The wax coated specimen is then placed in the cradle of special type balance. The cradle is then dipped in water contained in the bucket placed just below, and the apparent mass ( $M_3$ ) of waxed specimen is found.

From Archimedes' principle,  $M_3 = M_2 - V_w \rho_w$

or 
$$V_w = \frac{M_2 - M_3}{\rho_w}$$

Substituting the value of  $V_w$  in Eq. 3.40, we get.

$$V = \frac{M_2 - M_3}{\rho_w} - \frac{M_2 - M_1}{G_p} \quad \dots(3.40 \text{ a})$$

Thus, the volume of the specimen is known. The bulk density is then found from the relation.  $\rho = M_1/V$ .

**5. Rubber balloon method.** In this method, the volume of the excavated hole is measured with the help of an inflated rubber balloon. The apparatus consists of (i) a graduated

glass or lucite cylinder enclosed in an air-tight aluminium case, with an opening in the bottom, and (ii) a tray with central circular hole of 10 cm diameter (Anon : 1941). The cylinder is partially filled with water. Pressure or vacuum can be applied to the bottom of the cylinder with the help of a double acting rubber bulb.

The ground surface, where the density is to be determined is cleaned and levelled, and tray is placed over it. The cylinder is then placed centrally over the tray. The air valve is opened and air is pumped into the cylinder until the balloon is completely inflated against the surface of the soil in the opening of the tray. The water level is read in the cylinder. The cylinder is then removed, and a hole is excavated in the ground. The excavated soil is weighed, and a sample is kept for water content determination. The cylinder is then placed over the opening in the tray, air valve is opened and air is forced in the cylinder to inflate the bottom, until the base of the instrument is raised off the tray at least by 1 cm. The air valve is closed and both feet are placed firmly on the base plate so that the balloon is forced into any irregularities in the hole. The water level is read in the cylinder. The volume of the hole is found from the difference between the initial and final water level, in the glass cylinder. Knowing the mass, volume and water content, the bulk density and dry density can be computed.

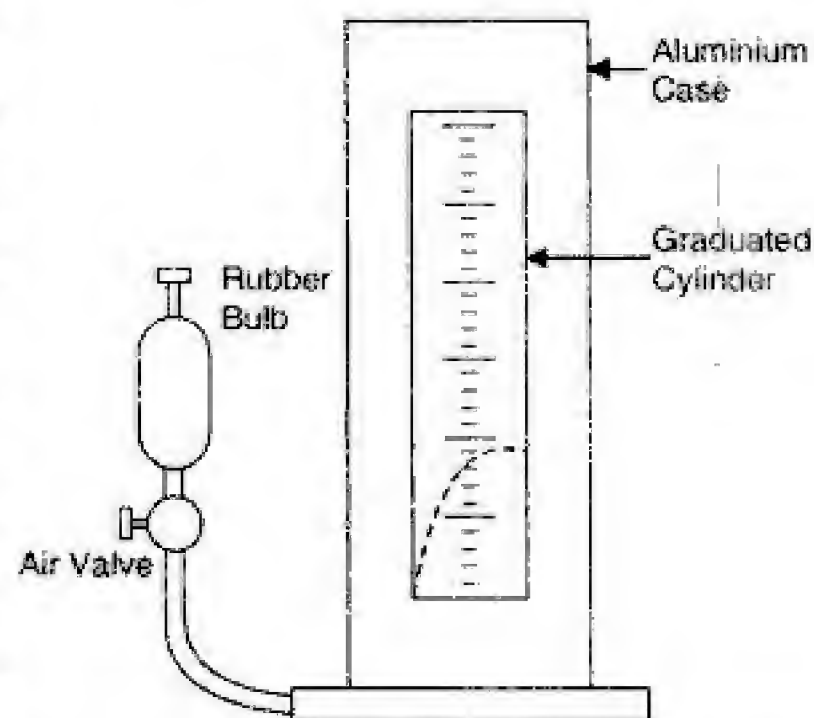


FIG. 3.21. RUBBER BALLOON CYLINDER.

**Void ratio and density index.** The void ratio of the soil in-situ can be computed from the following expression if  $G$  and  $\gamma_d$  are known :

$$e = \frac{G \gamma_w}{\gamma_d} - 1$$

The density index can then be calculated from the expression ;

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

To find  $e_{max}$ , the minimum density of the soil, corresponding to its loosest state, is found by pouring soil in a cylinder gently the help of a funnel. Similarly,  $e_{min}$  is found by compacting the soil by vibrating it to get its maximum density.

### 3.14. ACTIVITY OF CLAYS

The properties of clays and their behaviour is influenced by presence of certain clay minerals (see chapter 5) even in small quantities. The thickness of the oriented water around a clay particle is dependant on type of clay mineral. Thus, the plasticity of a clay depends upon (i) the nature of clay mineral present and (ii) amount of clay mineral present. On the basis of lab. tests, Skempton (1953) observed that *for a given soil* the plasticity index is directly proportional to the percent of clay-size fraction (*i.e.* percent by weight finer



than 0.002 mm in size). He introduced the concept of *activity*, by relating the plasticity to the quantity of clay-size particles, and defined the *activity* ( $A_c$ ) as the ratio of plasticity index to the percent by weight of soil particles of diameter smaller than two microns present in the soil. Thus

$$A_c = \frac{I_p}{C_w} \quad \dots(3.41)$$

where  $I_p$  = Plasticity index  
 $C_w$  = percentage, by weight of 'clay sizes',  
 i.e. of particles of size less than  
 2 microns.

Activity can be determined from the results of usual laboratory tests such as wet analysis, liquid limit and plastic limit.

Based on activity number, clays can be classified *qualitatively* into three categories as given in the Table 3.4 (also see Fig. 3.22).

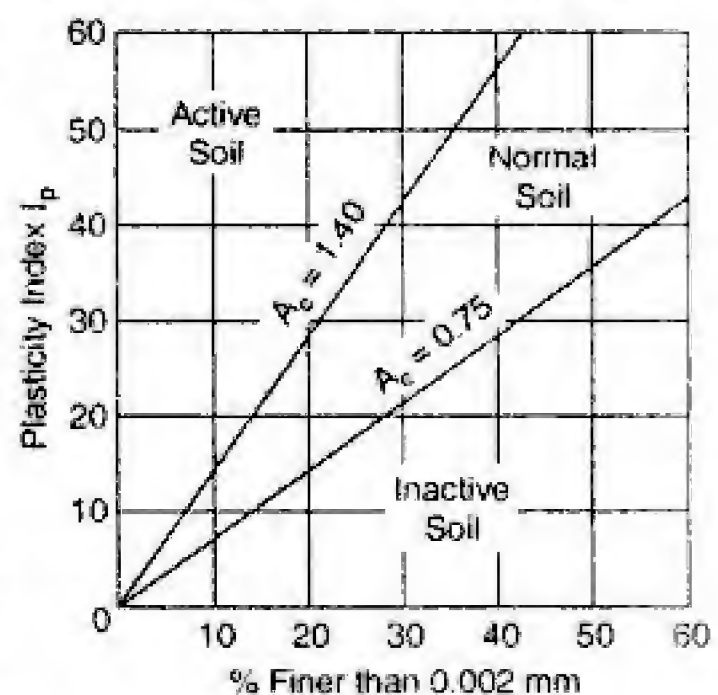


FIG. 3.22.

TABLE 3.4 : CLASSIFICATION BASED ON ACTIVITY

Activity	Classification
< 0.75	Inactive
0.75 – 1.40	Normal
> 1.40	Active

It should be noted that the activity of a given soil will be a function of type of clay mineral present in it. Clays containing kaolinite will have relatively low activity while those having montmorillonite will have high activity. Based on Eq. 3.41, typical activity values are as follows :

Kaolinite	0.4 – 0.5
Illite	0.5 – 1.0
Montmorillonite	1.0 – 7.0

### 3.15. SENSITIVITY OF CLAYS

The consistency of an undisturbed sample of clay is altered, even at the same water content, if it is remoulded. It is because the original structure of clay is altered by reworking or remoulding. Since the strength of a clay soil is related to its structure, remoulding results in decrease of its strength. The degree of disturbance of undisturbed clay sample due to remoulding is expressed by *sensitivity* ( $S_r$ ) which is defined as the ratio of its unconfined compression strength in the natural or undisturbed state to that in the remoulded state, without change in the water content :

$$S_t = \frac{q_u \text{ (undisturbed)}}{q_u \text{ (remoulded)}} \quad \dots(3.42)$$

The sensitivity of most clays generally falls in a range of 1 to 8. Table 3.5 below gives the classification of clays based on the sensitivity. Clays having sensitivity greater than 16 are known as *quick* clays. Highly overconsolidated clays tend to be insensitive

TABLE 3.5 SENSITIVITY CLASSIFICATION

Sensitivity	Classification	Structure
1	Insensitive	—
2 to 4	Normal or less sensitive	Honey comb structure
4 to 8	Sensitive	Honey comb or Flocculent structure
8 to 16	Extra sensitive	Flocculent structure
> 16	Quick	Unstable.

### 3.16. THIXOTROPY OF CLAYS

When sensitive clays are used in construction, they lose strength due to remoulding during construction operations. However, with passage of time, the strength again increases, though not to the same original level. This phenomenon of 'strength loss-strength gain' with no change in volume or water content is called '*thixotropy*' (from the Greek *thixis*, meaning 'touch' and *tropein*, meaning 'to change'). Thus, *thixotropy* is defined as an isothermal, reversible, time dependent process which occurs under constant composition and volume, thereby a material softens, as a result of remoulding, and then gradually returns to its original strength when allowed to rest. This is shown in Fig. 3.23. The larger the sensitivity, larger the *thixotropic hardening*.

The loss of strength due to remoulding is partly due to (i) permanent destruction of the structure due to in-situ layers and (ii) reorientation of the molecules in the adsorbed layers. *The gain in strength is due to rehabilitation of the molecular structure of the soil*, and is due to its thixotropic property.

The loss of strength due to *destruction* of structure cannot be recouped with time. The regaining of a part of the strength after remoulding has important application in connection with pile driving operations, and other types of construction in which disturbance of natural clay formations is inevitable.

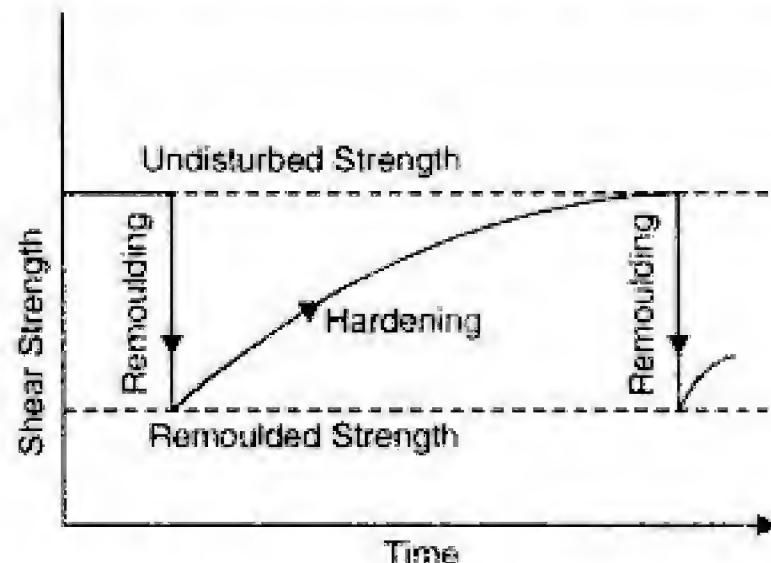


FIG. 3.23 THIXOTROPY OF CLAYS

### 3.17. COLLAPSIBLE SOILS

*Collapsing or collapsible soils* are those soils which undergo large decrease in volume due to increase in moisture content, even without increase in external loads. Examples of soils exhibiting this behaviour are loess, weakly cemented sands and silts where cementing agent is soluble (e.g. soluble gypsum, halite etc.) and certain granite residual soils. The Authors have observed some dune sands exhibiting such behaviour. A common feature of

collapsible soils are the loose bulky grains held together by capillary stresses. Deposits of collapsible soils are usually associated with regions of moisture deficiency, such as those in arid and semi-arid regions.

Potentially collapsible soils could be identified and classified by detailed geologic studies. Fig 3.24. (Holtz and Gibbs, 1967) provides guidance for identifying the potential for collapse for clayey sands and sandy clays found in western U.S.

The potential for collapse is expressed by the term *collapse potential* (CP) or *coefficient of structural collapse* defined by

$$CP = \frac{\Delta e_c}{1 + e_1} \quad \dots(3.43)$$

or

$$CP = \frac{\Delta H_c}{H_1} \quad \dots(3.43 \text{ a})$$

where  $\Delta e_c$  = change in void ratio upon wetting

$e_1$  = voids ratio, before saturating or wetting the soil.

$\Delta H_c$  = change in height upon wetting ;  $H_1$  = initial height

The sudden volume decrease due to collapse of structure can be easily studied in the laboratory by consolidometer test conducted on special oedometer devised by Vasiliev (1949). During the test, the natural specimen is first consolidated under loads increased in steps, and a normal  $p$ - $e$  curve is obtained (Fig 3.25). At some pre-fixed load intensity, after the consolidation is complete, the specimen is wetted through porous stones with enough water to become fully saturated. This will result in a sudden compression ( $\Delta e_c$ ) which can be observed.

At given over-burden pressure ( $p_o$ ), the collapse potential depends upon the degree of saturation ( $S$ ) as shown in Fig 3.26. The bulk of structural collapse usually occurs upon the first saturation, and a repeated submersion produces only a very small further decrease in volume. The collapse potential (CP) varies with the pressure as shown in Fig 3.27. The reduction in volume is a maximum at a pressure which is usually in the vicinity  $300 \text{ kN/m}^2$ .

Denisov (1946) used the ratio  $e_L/e_o$  to characterise the collapsibility of a soil, where  $e_L$  is the voids ratio corresponding to liquid limit and  $e$  is the natural voids ratio. If this ratio is greater than unity, the soil is described as 'collapsible'

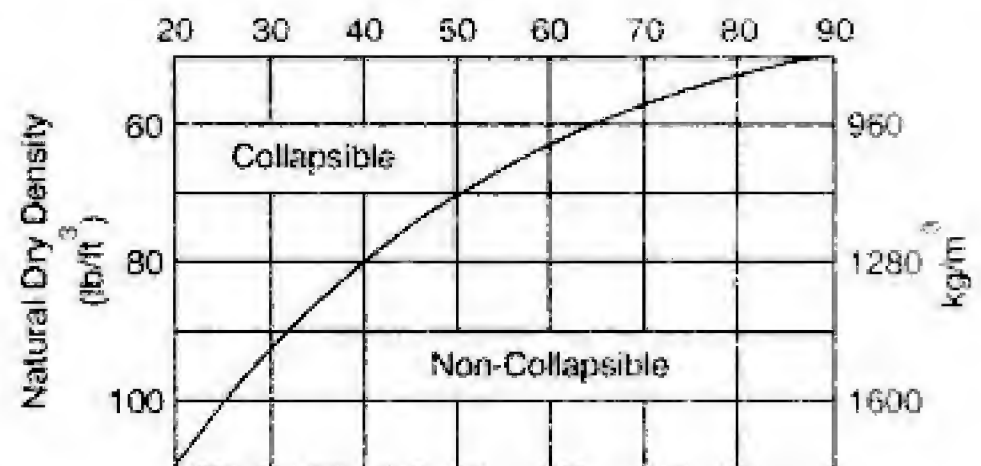


FIG 3.24. CRITERION FOR COLLAPSE POTENTIAL

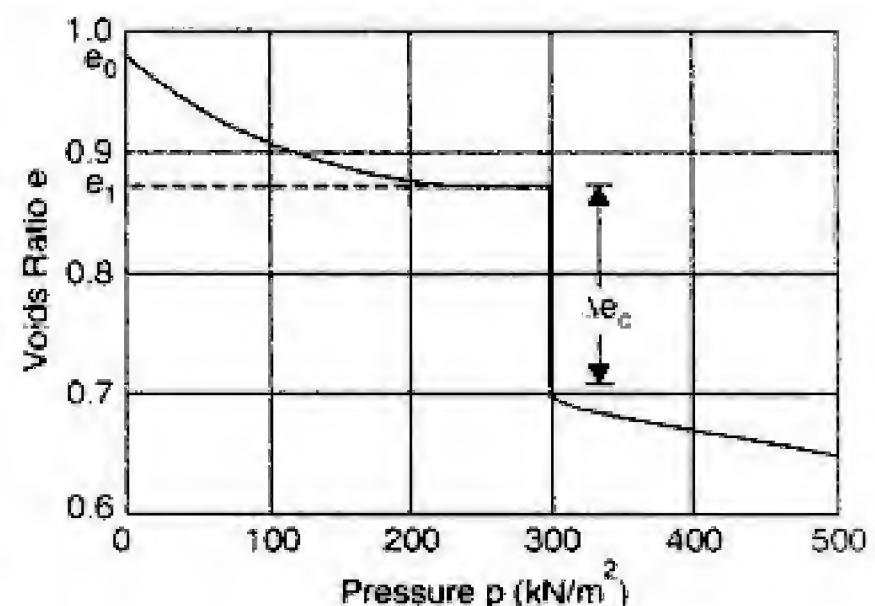
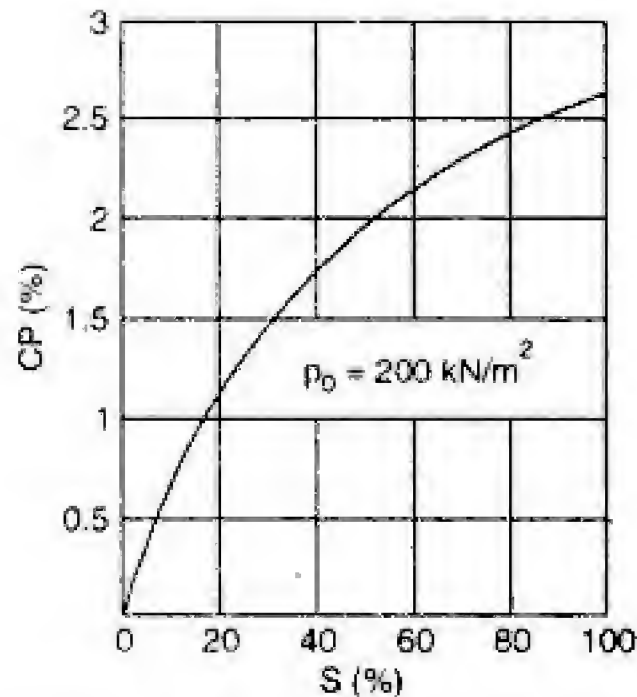
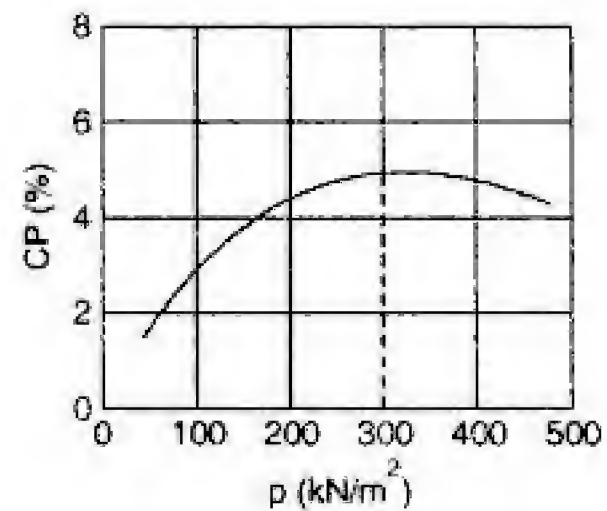


FIG. 3.25 'COLLAPSE POTENTIAL



FIG. 3.26. EFFECT OF  $S$  ON CP.FIG. 3.27. EFFECT OF  $p$  ON CP.

The potential for collapse can also be evaluated in the field by performing standard plate load tests under varied moisture environments.

Table 3.6 below gives the effect of collapse potential values on the severity of problem.

TABLE 3.6 : COLLAPSE POTENTIAL VALUES

CP	Severity of Problem
0 - 1 %	No problem
1 - 5 %	Moderate trouble
5 - 10 %	Trouble
10 - 20 %	Severe trouble
> 20 %	Very severe trouble.

**Example 3.12.** A clay sample has liquid limit and plastic limit of 96% and 24% respectively. Sedimentation analysis reveals that the clay soil has 50% of the particles smaller than 0.002 mm. Indicate the activity classification of the clay soil and the probable type of clay mineral.

**Solution :** We have  $w_L = 96\%$  and  $w_P = 24\%$

Hence plasticity index,  $I_P = w_L - w_P = 96 - 24 = 74\%$

Now, from Eq. 3.41, activity  $A_C = \frac{I_P}{C_w} = \frac{74}{50} = 1.48$

Since the activity No. is greater than 1.4, clay may be classified as being active. Also, the probable clay mineral is *montmorillonite*

**Example 3.13.** A clay specimen has unconfined compressive strength of 240 kN/m<sup>2</sup> in undisturbed state. Later, on remoulding, the unconfined compressive strength is found to be 54 kN/m<sup>2</sup>. Classify the clay soil on the basis of sensitivity and indicate the probable structure of clay soil.

**Solution :** From Eq. 3.42. Sensitivity,  $S_r = \frac{q_u \text{ (undisturbed)}}{q_u \text{ (disturbed)}} = \frac{240}{54} = 4.44$

Since  $S_r$  is between 4 and 8, the given clay is classified as 'sensitive'. The possible structure of clay soil may be *honey-comb* or *flocculent*.

### 3.18. AVERAGE DIAMETER OF A GROUP OF PARTICLES

A given mass of granular soil has particles of different sizes. Some times, it may be necessary to state the *average diameter* of the group of particles present in the soil mass. Following are the most common ways to interpret average diameter ( $D_{av}$ ), along with corresponding expressions used for the determination.

(a) *Arithmetic mean method*

$$D_a = \frac{\sum_{i=1}^J n_i D_i}{\sum_{i=1}^J n_i} \quad \dots(3.44 \ a)$$

(b) *Geometric mean method*

$$D_g = (D_1 D_2 D_3 \dots D_n)^{1/J} \quad \dots(3.44 \ a)$$

(c) *Harmonic mean method*

$$D_h = \frac{1}{\sum_{i=1}^J \frac{n_i}{D_i}} \quad \dots(3.44 \ c)$$

where

$D_a$  = average diameter by arithmetic mean method.

$D_g$  = average diameter by geometric mean method.

$D_h$  = average diameter by harmonic mean method.

Example 3.13 Illustrates the method of computing average diameter of a group of particles in a hypothetical soil mass, using the above equations.

Out of the three mean diameters ( $D_a$ ,  $D_g$  and  $D_h$ ) the average diameter by harmonic method (i.e.  $D_h$ ) is quite important since it is sometimes used to compute the *specific surface*. The harmonic mean diameter ( $D_h$ ) can be obtained from the following expression assuming a continuous distribution [ $P = f(D)$ ] :

$$\frac{1}{D_h} = \int_0^1 \frac{dP}{D} \quad \dots(3.45)$$

Kozney (1931) developed a simple graphical method for determining  $D_h$ , by plotting two sets of particle size distribution curves : (i) curve between percentage finer  $N$  and  $\log D$ , and (ii) curve between percent finer  $N$  and  $1/D$ . The method of finding  $D_h$  from these two set of curves has been explained numerically in Example 3.15.

### 3.19. SPECIFIC SURFACE

*Specific surface* is defined as the total surface area of the soil particles in a unit mass. In order to understand and appreciate the physical significance of this term, let us consider a cube of soil particle having 10 mm side. The surface area of each face of

the cube is  $10 \times 10$  sq. mm and hence the total surface area of the cube  $= 6 \times 10 \times 10 = 600$  sq. mm. Let us now subdivide this cube into 8 cubes, each with 5 mm side. The total surface area of the same original cube, subdivided into eight smaller cubes will be  $= 8 \times 6 \times 5 \times 5 = 1200$  sq. mm. If the same cube is further subdivided into smaller cubes each with sides of  $1 \mu m$ , then the number of small cubes will be  $10^{12}$  and their total surface area will be  $10 m^2$  ( $10^7 mm^2$ ). Thus the subdivision of the same mass yields greater surface area. *Hence specific surface increases very rapidly as the grain size decreases.* Microscopic particles have very large specific surface.

Another factor which is very important for the magnitude of specific surface is the *grain shape*. Let us consider soil solids of various shapes, all having a constant volume  $V = 1 cm^3$  ( $1000 mm^3$ ), and hence of constant mass. Their specific surfaces will be as under:

	Particle shape	Specific surface (per $cm^3$ )
1.	Spherical	$483 mm^2$
2.	Cubical	$600 mm^2$
3.	Disc, 1 mm thick	$2110 mm^2$
4.	Plate, $1 \mu m$ thick	$2 \times 10^6 mm^2$

The specific surface per unit mass is of particular importance in the study of fine grained soils, since the electrical forces responsible for inter - particle effects are proportional to the specific surface (see chapter 5). The chemical activity of the clay minerals depends upon the surface area, because the electrical charge of clay minerals is high per unit mass. Permeability of soil greatly depends on the specific surface of the soil because of the viscous effects associated with surface characteristics of the grains. Specific surface is also related to many other physical properties of soils.

**Expression for specific surface for soil having spherical particles :** Let us consider unit mass of soil having spherical particles of various sizes. The *specific surface* for such a soil can be determined in terms of its *harmonic mean diameter* ( $D_h$ ) defined by equation 3.44 (c). If there are  $n$  number of particles of mean diameter  $D_h$  in that unit soil mass, having mass density  $\rho_s$  of the soils, we have

$$n \left( \frac{\pi}{6} D_h^3 \rho_s \right) = \text{unit mass} = 1. \quad \dots(1)$$

Also, if  $F$  is the *specific surface* (i.e. the total surface area of the particles on a unit mass), we have  $F = n (\pi d)$  ...(2)

Substituting the value of  $n$  from (1) into (2), we get

$$F = \frac{6}{\rho_s} \cdot \frac{1}{D_h} \quad \dots(3.45 a)$$

The harmonic mean diameter  $D_h$  can be found by Kozney's method explained in § 3.19 and illustrated in Example 3.14

*Specific surface* ( $S_s$ ) is also expressed as surface area of the particle per unit volume of the particle, i.e.



$$\text{Specific surface} \quad S_s = \frac{\text{Surface area of particle}}{\text{Volume of particle}} = \frac{\pi D^2}{\frac{1}{6} \pi D^3} = \frac{6}{D} \quad \dots(3.45)$$

For spheres uniformly distributed in size between the mesh sizes  $x$  and  $y$  of the adjacent sieves is given by  $S_s = \frac{6}{\sqrt{x \cdot y}} \quad \dots(3.45 \text{ b})$

The above equation gives results accurate to 2% if  $x/y$  is not greater than 2 and the size distribution by mass is logarithmically uniform. The following table gives the specific surface of spheres lying between the BS sieves/equivalent IS sieves.

Equivalent IS sieves	Specific surface ( $\text{cm}^2/\text{cm}^3$ )
2.36 – 1.18 mm	35.2
1.18 mm – 600 (microns)	70.6
600 – 300 (microns)	143.0
300 – 150 (microns)	284.0
150 – 75 (microns)	558.0

**Example 3.14.** Columns 1 and 2 of the table below give the limits of the particle size and the number of the particles respectively of a given soil mass. Determine arithmetic mean diameter, geometric mean diameter and harmonic mean diameter of the group of particles. Taking density of soil solids as  $2.72 \text{ g/cm}^3$ , determine the specific surface of the group of particles.

**Solution**

1	2	3	4	5
Limits of the particles size ( $\mu$ )	No. of particles ( $n$ )	Average size $D_i$ ( $\mu$ )	$nD$	$n/D$
0 – 6	3	3	9	1
6 – 12	12	9	108	1.33
12 – 18	60	15	900	4.0
18 – 24	85	21	1785	4.05
24 – 30	30	27	810	1.11
30 – 36	20	33	660	0.61
36 – 42	16	39	624	0.41
42 – 48	8	45	360	0.18
48 – 54	4	51	204	0.08
54 – 60	2	57	114	0.04
sum	240		5574	12.01

Hence,  $J = \text{No. of groups} = 10$

$$\text{Now, } D_a = \frac{\sum_{i=1}^J n_i D_i}{\sum_{i=1}^J n_i} = \frac{5574}{240} = 23.23 \mu = 23.23 \times 10^{-3} \text{ mm}$$

$$D_g = (D_1 D_2 D_3 \dots D_n)^{1/n} = (3 \times 9 \times 15 \times 21 \times 27 \times 33 \times 39 \times 45 \times 51 \times 57)^{1/10}$$

$$= 22.84 \mu = 22.84 \times 10^{-3} \text{ mm}$$

$$\frac{1}{D_h} = \frac{1}{\sum_{i=1}^J n_i} \frac{n_i}{D_i} = \frac{1}{240} \quad (12.01) \quad \text{or} \quad D_h = \frac{240}{12.01} = 19.98 \mu = 19.98 \times 10^{-3} \text{ mm}$$

**Example 3.15.** The particle size analysis of a soil sample gave the following results.

Particle size (mm)	% finer (N)	Particle size (mm)	% Finer (N)
0.2	100	0.01	19
0.1	95	0.005	9
0.05	89	0.002	4
0.02	40	0.001	2

Using Kozney's graphical method, determine the harmonic mean diameter,  $D_h$ .

**Solution**

**Step 1 :** Plot the usual particle size distribution curve between N and  $\log_{10} D$ . (curve I of Fig. 3.28).

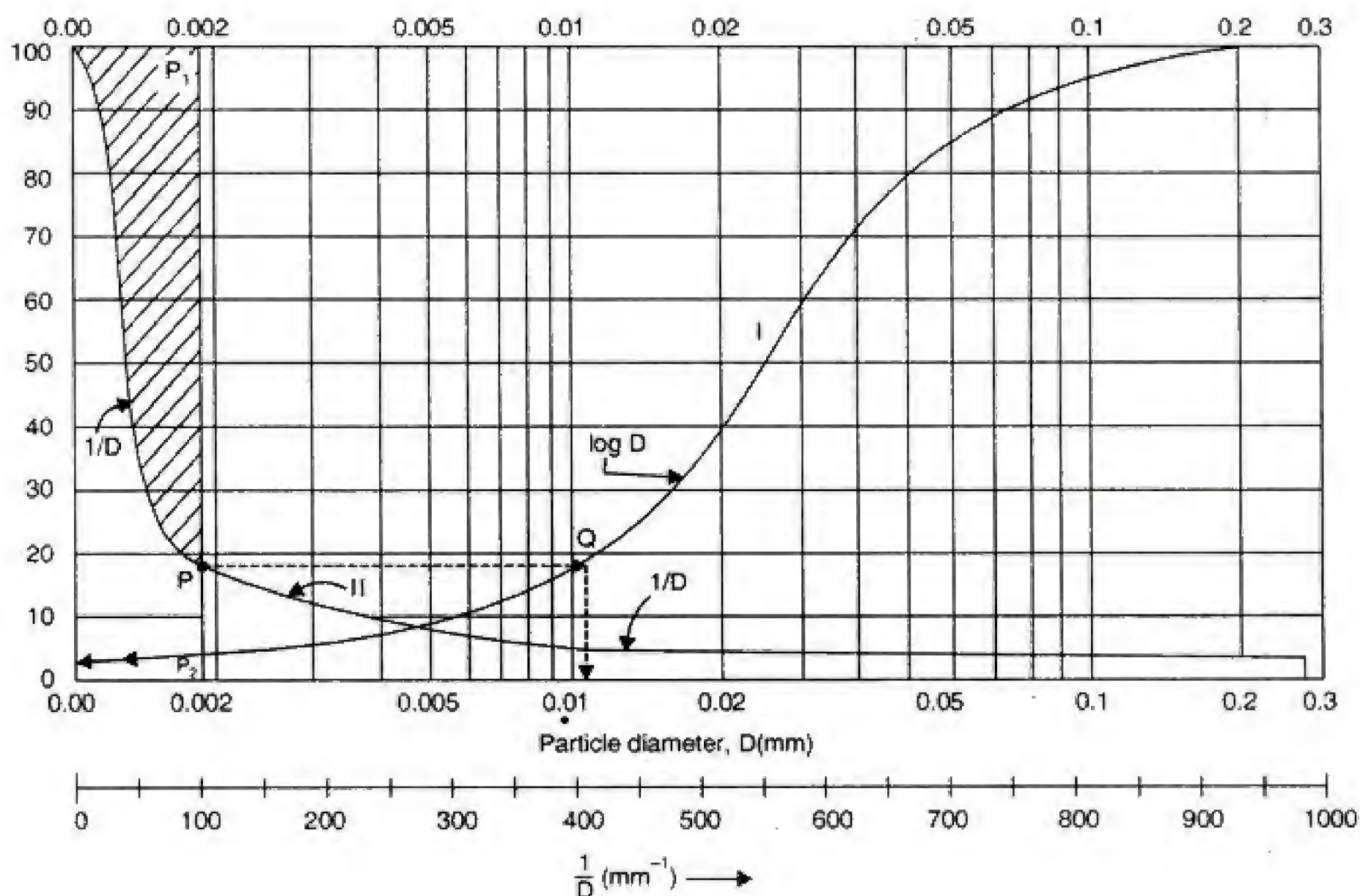


FIG. 3.28

**Step 2 :** From the given data, derive a table between  $1/D$  and  $N$ , as follows

$1/D \text{ mm}^{-1}$	$N (\%)$	$1/D \text{ mm}^{-1}$	$N\%$
5	100	100	19
10	95	200	9
20	89	500	4
50	40	1000	2

**Step 3 :** On the same sheet, plot a curve between  $1/D$  and  $N$  (curve II of Fig. 3.28).

**Step 4 :** Convert the area below curve II into a rectangle of height  $N = 100\%$ . This means that a vertical line  $P_1 P_2$  is drawn in such a way that the two shaded areas, shown in Fig. 3.28 are equal. This is done by trial.

**Step 5:** The vertical line  $P_1 P_2$  will cut the curve II in point  $P$  which corresponds to the harmonic mean diameter  $D_h$ . Draw horizontal line  $PQ$  to intersect curve I in the point  $Q$ . Project point  $Q$  downwards on the log scale to get the required harmonic mean diameter  $D_h$ .

Thus, from Fig. 3.28, we get  $D_h = 0.01 \text{ mm}$

### 3.20. EXAMPLES FROM COMPETITIVE EXAMINATIONS

**Example 3.16.** The plastic limit of a soil is 25% and its plasticity index is 8%. When the soil is dried from its state of plastic limit, the volume change is 25% of its volume at plastic limit. Similarly, the corresponding volume change from liquid limit to dry state is 34% of its volume at liquid limit. Determine the shrinkage limit and shrinkage ratio. (Civil Services Exam. 1989)

**Solution :** This question was set from Author's present book. See example 3.9 for detailed solution.

**Answers :**  $w_s = 10.4\%$  and  $SR = 2.27$

**Example 3.17.** The plastic limit of a soil is 24% and its plasticity index is 80%. When the soil is dried from its state at plastic limit, the volume change is 26% of its volume of plastic limit. The corresponding volume change from liquid limit to dry state is 35% of its volume at liquid limit. Determine the shrinkage limit and the shrinkage ratio. (Civil Services Exam, 1993)

**Solution :** This question was set on the basis of example 3.9 of this book, simply by reducing all the volumes by 1%. Otherwise the question is exactly the same. Hence refer example 3.9 for the solution of this question.

Given :  $w_p = 24\%$ ,  $I_p = 8\%$  ;

$$\therefore w_L = 24 + 8 = 32\%$$

Volume change at liquid limit = 35% ; Volume change at plastic limit = 26%

$\therefore$  Dry Volume,

$$V_d = V_L - 0.35 V_L = 0.65 V_L \quad \dots(i)$$

$$\text{Also, } V_d = V_p - 0.26 V_p = 0.74 V_p \quad \dots(ii)$$

$$\therefore \text{ From (i) and (ii), } V_p = \frac{0.65}{0.74} V_L$$

$$= 0.8784 V_L \text{ . From Fig. 3.29,}$$

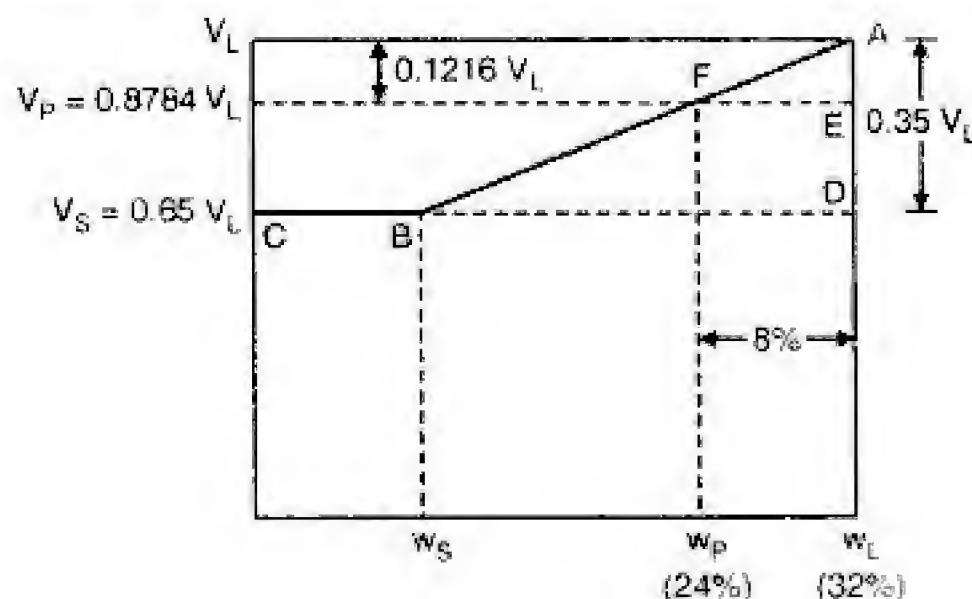


FIG. 3.29



$$\frac{BD}{AD} = \frac{FE}{AE} = \frac{8}{0.1216 V_L} \quad \therefore \quad BD = \frac{8}{0.1216 V_L} \times 0.35 V_L = 23.02\%$$

$$\therefore \quad w_s = w_L - 23.02 = 32 - 23.02 = 8.98\%$$

$$SR = \frac{(V_1 - V_2)/V_d}{w_1 - w_2} = \frac{(V_L - V_P) 100}{V_d (w_L - w_P)} = \frac{V_L (1 - 0.8784)}{0.65 V_L (32 - 24)} = 2.338$$

**Example 3.18.** The plastic limit and liquid limit of a soil are 30% and 42% respectively. The percentage volume change from the liquid limit to dry state is 35% of the dry volume. Similarly, the percentage volume change from the plastic limit to the dry state is 22% of the dry volume. Determine the shrinkage limit and the shrinkage ratio. (Civil Services Exam, 1997)

**Solution :** This question is similar to example 3.9 except that volume changes are corresponding to dry weights. Given :  $w_L = 42\%$  ;  $w_P = 30\%$  ;  $V_L - V_d = 0.35 V_d$  or  $V_L = 1.35 V_d$

Also,  $V_P - V_d = 0.22 V_d$  or

$$V_P = 1.22 V_d$$

From the geometry of Fig. 3.30

$$\frac{V_L - V_P}{w_L - w_P} = \frac{V_L - V_d}{w_L - w_s}$$

$$\text{or } \frac{1.35 V_d - 1.22 V_d}{0.42 - 0.30} = \frac{1.35 V_d - V_d}{0.42 - w_s}$$

From which

$$w_s = 0.42 - \frac{1.35 - 1}{1.35 - 1.22} \times 0.12$$

$$= 0.0969 = 9.69\%$$

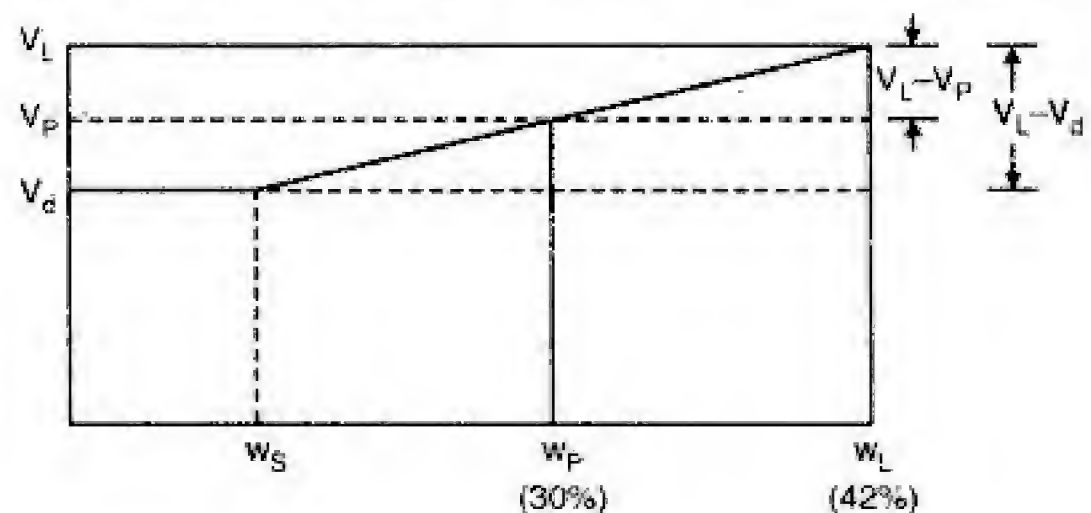


FIG. 3.30

$$\text{Also, } SR = \frac{(V_1 - V_2)/V_d}{w_1 - w_2} = \frac{V_L - V_P}{V_d (w_L - w_P)} = \frac{1.35 - 1.22}{0.42 - 0.30} = 1.083$$

**Example 3.19.** The mass specific gravity of a fully saturated specimen of clay having a water content of 40% is 1.88. On oven drying the mass specific gravity drops to 1.74. Calculate the specific gravity of clay and its shrinkage limit. (Engg. services Exam, 1992)

**Solution :** The above problem is based on Example 3.7

Given,  $G_m, sat = 1.88$  ;  $w = 40\%$  ,  $G_m, dry = 1.74$

$$\text{For saturated soil } G_m = \frac{\gamma_{sat}}{\gamma_w} \text{ or } \gamma_{sat} = G_m \cdot \gamma_w = 1.88 \times 9.81 = 18.443 \text{ kN/m}^3$$

$$\text{Now, } \gamma_{sat} = \frac{(G + e) \gamma_w}{1 + e}, \text{ where } e = e_{sat} = w_{sat} G = 0.4 G$$

$$\therefore \gamma_{sat} = 18.443 = \frac{(G + 0.4 G)}{1 + 0.4 G} \cdot \gamma_w = \frac{1.4 G}{1 + 0.4 G} \times 9.81 \text{ or } \frac{1.4 G}{1 + 0.4 G} = \frac{18.443}{9.81} = 1.88, \text{ or } G = 2.901$$

$$G_m, dry = 1.74 = \frac{\gamma_d}{\gamma_w} ; \quad \text{Now } w_s = \frac{\gamma_w}{\gamma_d} - \frac{1}{G} = \frac{1}{1.74} - \frac{1}{2.901} = 0.23 = 23\%$$

**Example 3.20.** The values of liquid limit, plastic limit and shrinkage limit of a soil were reported as :  $w_L = 60\%$  ,  $w_P = 30\%$  ,  $w_s = 20\%$

If a sample of this soil at liquid limit has a volume of 40 c.c. and its volume measured at shrinkage limit was 23.5 c.c., determine the specific gravity of the solids. What is the shrinkage ratio and volumetric shrinkage ? (Engg. Services Exam, 1996)

**Solution :** This problem is exactly similar to example 3.8.

$$\Delta M_w \text{ between (a) and (b)} = 0.6 M_d - 0.2 M_d = 0.4 M_d$$

...(i)

Now,  $V_1 - V_2 = 40 - 23.5 = 16.5 \text{ cm}^3$ . Hence  $\Delta M_w = 16.5 \times \rho_w = 16.5 \text{ g}$  ... (ii)

From (i) and (ii), we get  $M_d = 16.5 / 0.4 = 41.25 \text{ g}$

$\therefore M_w \text{ in (a)} = 0.2 M_d = 0.2 \times 41.25 = 8.25 \text{ g}$

$\therefore V_w \text{ in (b)} = M_w / \rho_w = 8.25 / 1 = 8.25 \text{ cm}^3$

$$V_s = V_2 - V_w = 23.5 - 8.25 = 15.25 \text{ cm}^3$$

Hence  $\rho_s = \frac{M_d}{V_s} = \frac{41.25}{15.25} = 2.705$  ; Hence  $G_s = G = \frac{\rho_s}{\rho_w} = \frac{2.705}{1} = 2.705$

Shrinkage ratio  $SR = \frac{\rho_d}{\rho_w} = \frac{M_d / V_d}{1} = \frac{41.25}{23.5} = 1.755$

Also, volumetric shrinkage  $VS = \frac{V_1 - V_d}{V_d} = \frac{40 - 23.5}{23.5} = 0.7201 = 72.01\%$

Check :  $VS = (w_1 - w_2) SR = (0.6 - 0.2) \times 1.755 = 0.7021$

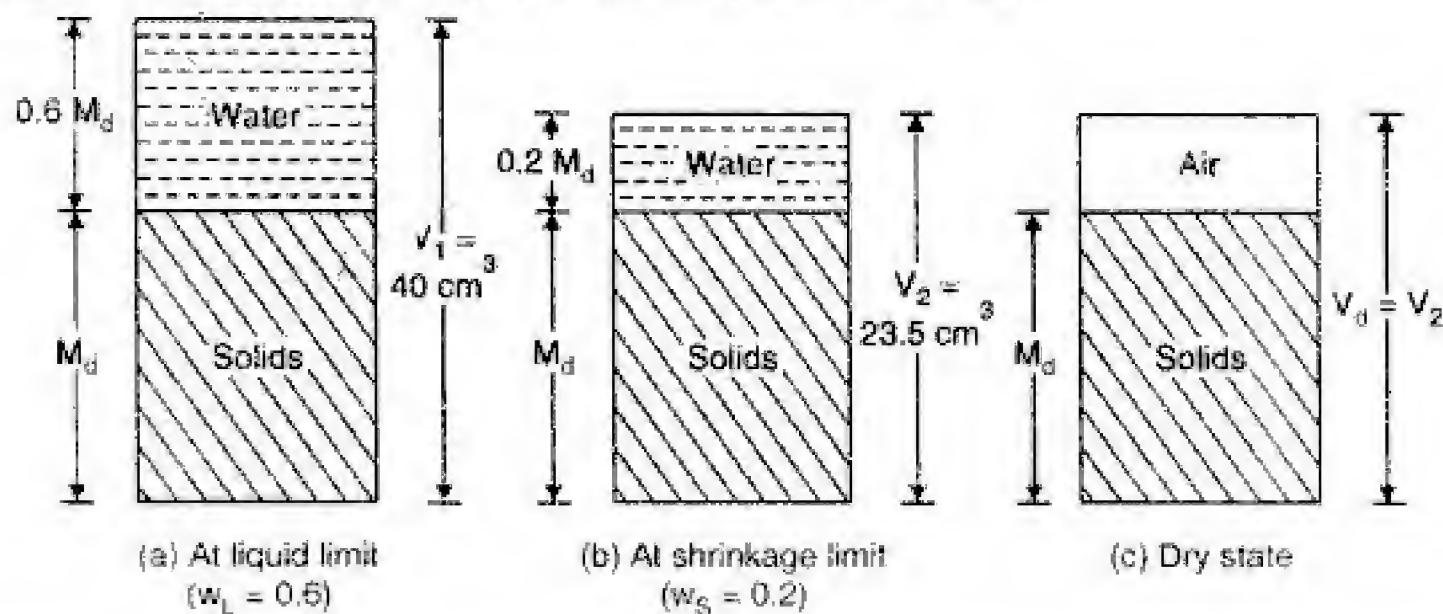


FIG. 3.31

**Example 3.21.** A soil deposit has a void ratio of 0.9. If the void ratio is reduced to 0.6 by compaction, find the percentage volume loss due to compaction. (Gate Exam. 1988)

**Solution:** Given :  $e_1 = 0.9$  ;  $e_2 = 0.6$ . Let us use symbol 1 for initial condition and 2 for finally compacted condition.

Initially  $\frac{V_{v1}}{V_s} = e_1 = 0.9$  ; Hence  $V_{v1} = 0.9 V_s$   $\therefore V_1 = V_{v1} + V_s = 0.9 V_s + V_s = 1.9 V_s$  ... (1)

Finally  $\frac{V_{v2}}{V_s} = e_2 = 0.6$  ; Hence  $V_{v2} = 0.6 V_s$   $\therefore V_2 = V_{v2} + V_s = 0.6 V_s + V_s = 1.6 V_s$  ... (2)

$$\therefore \% \text{ loss of volume} = \frac{1.9 V_s - 1.6 V_s}{1.9 V_s} \times 100 = 15.79\%$$

### 3.21. LABORATORY EXPERIMENTS

#### EXPERIMENT 1 : DETERMINATION OF WATER CONTENT BY OVEN-DRYING METHOD

**Object and scope.** The object of this test is to determine the water content of a soil sample in the laboratory by oven-drying method. This experiment forms an essential part of many other laboratory experiments.

**Material and equipment.** (i) Non-corrodible air-tight containers, (ii) Balance of sufficient sensitivity to weigh soil samples to an accuracy of 0.04 per cent of the mass of the soil taken for the test. For fine grained soils the balance should have an accuracy of 0.01 g and for coarse-grained soil, it should have an accuracy of 0.1 g, (iii) Desiccator with any suitable desiccating agent, (iv) Thermostatically

controlled oven, with interior of non-corroding material, to maintain temperature between 105 °C to 110 °C.

#### Test Procedure

1. Clean the container and find its mass with lid ( $M_1$ ).
2. Put the required quantity of the moist soil sample in the container and replace the lid. Take the mass  $M_2$ . The quantity of the soil sample to be taken depends upon its gradation and maximum size of particles. The following quantities are recommended for laboratory use:

<i>Size of particles more than 90% passing</i>	<i>Minimum quantity of soil specimen to be taken for testing (g)</i>
425-micron IS sieve	25
2 mm IS sieve	50
4.75 mm IS sieve	200
9.5 mm IS sieve	300
19 mm IS sieve	500
37.5 mm IS sieve	1000

3. Keep the container in the oven with lid removed and maintain the temperature of the oven between 105 °C to 110 °C, for about 16 to 24 hours.

4. Take out the container, replace the lid and cool it in the desiccator. Find the mass  $M_3$  of the container with lid and dried soil sample.

**Tabulation of observations.** The results of the test are recorded as illustrated in Table 3.7.

**TABLE 3.7 : DATA AND OBSERVATION SHEET FOR WATER  
CONTENT DETERMINATION [IS : 2720 (PART II)—1973]**

Sample No....

Tested by.....

1.	Container No.		42	
2.	Mass of container and wet soil	$M_2$ g	50.21	
3.	Mass of container and dry soil	$M_3$ g	48.05	
4.	Mass of container	$M_1$ g	20.42	
5.	Mass of dry soil	$(M_3 - M_1)$ g	27.63	
6.	Mass of water	$(M_2 - M_3)$ g	2.16	
7.	Water content	$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100\%$	7.8	

**Calculations.** The water content of the sample is calculated from the following expression (Eq.3.1)

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \text{ (per cent)}$$

**Precautions and discussions.** (i) Oven-drying at 105 °C to 110 °C does not result in reliable water content values for soil containing gypsum or other minerals having loosely bound water of hydration or for soil containing significant amounts of organic material. Reliable water content values for these soils can be obtained by drying in oven at approximately 60 °C to 80 °C. (ii) The specimen should be dried in the oven to constant mass indicated by the difference between two consecutive mass of the container with the dried specimen, taken at suitable intervals after initial drying, being a maximum of 0.1 per cent of the original mass of the soil specimen.

**Referenece to Indian Standard.** IS : 2720 (Part II)—1973, Method of Test for Soils : Part II : Determination of Moisture Content.



**EXPERIMENT 2 : DETERMINATION OF SPECIFIC GRAVITY OF SOIL BY DENSITY BOTTLE**

**Object and scope.** The object of the test is to determine the specific gravity of soil fraction passing 4.75 mm sieve by density bottle.

**Materials and equipment.** (i) Density bottle of 50 ml or 100 ml capacity, with stopper having capillary hole at its centre, (ii) Constant temperature water bath maintaining a constant temperature of 27 °C , (iii) Balance sensitive to 0.001 g, (iv) Vacuum source, (v) Wash bottle filled with deaired distilled water, (vi) Alcohol, (viii) Ether.

**Test Procedure**

1. To clean and dry the density bottle, wash it thoroughly with distilled water and allow it to drain. Rinse the bottle with alcohol to remove the water and drain the alcohol. Then rinse the bottle with ether to remove the alcohol and drain the ether. Turn the bottle upside down for a few minutes to permit the ether vapour to come out.

2. Find the mass of the empty cleaned bottle ( $M_1$ ) accurate to 0.001 g with its stopper.

3. Take about 10 to 20 g of oven-dried soil sample, cooled in the desiccator and transfer it carefully to the density bottle. Find the mass ( $M_2$ ) of the bottle and the soil, with the stopper.

4. Put about 10 ml deaired distilled water in the bottle, so that the soil is fully soaked. Leave it for a period of 2 to 10 hours.

5. Add more distilled water so that the bottle is about half full. Remove the entrapped air by subjecting the contents to a partial vacuum (air pressure not exceeding 100 mm of mercury).

6. Fill in the bottle completely, put the stopper on and place the bottle on the stand fitted in the constant temperature water bath. Keep it there for about one hour so that the temperature of soil and water in the bottle also reaches 27 °C.

7. Take out the bottle and wipe it clean and dry from outside. Fill the capillary of the stopper with drops of distilled water, in case it is not full. Determine the mass of the bottle and its contents ( $M_3$ ) .

8. Empty the bottle and clean it thoroughly. Fill it with distilled water, put on the stopper and wipe the bottle dry from outside. Find the mass ( $M_4$ ) .

9. Repeat steps 3 to 8 and take two more determinations.

**Tabulation of observations.** The observations are tabulated as shown in Table 3.8

**TABLE 3.8 : DATA AND OBSERVATION SHEET FOR SPECIFIC GRAVITY  
DETERMINATION BY DENSITY BOTTLE**

Sample No. A-13

<i>Determination</i>	<i>1</i>	<i>2</i>	<i>3</i>
1. Density bottle No.	1	2	3
2 Mass of density bottle ( $M_1$ ) g	31.352	32.560	32.125
3. Mass of bottle + dry soil ( $M_2$ ) g	44.873	44.615	45.213
4. Mass of bottle + soil + water ( $M_3$ ) g	140.563	140.468	140.260
5. Mass of bottle + water ( $M_4$ ) g	132.052	132.875	132.024
Specific gravity ( $G$ )	2.705	2.700	2.695
Average specific gravity (at 27°C) = 2.70			

**Calculations.** The specific gravity of the soil is calculated from Eq. 3.3 :

$$G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$

**EXPERIMENT 3 : DETERMINATION OF SPECIFIC GRAVITY BY PYCNOMETER**

**Object and scope.** The object of the test is to determine the specific gravity of soil fraction passing 4.75 mm IS sieve by pycnometer.

**Materials and equipment.** (i) Pycnometer of about 900 ml capacity, with a conical brass cap and screwed at its top, (ii) Balance sensitive to 1 g, (iii) Glass rod, (iv) Deaired, distilled water.

**Test Procedure**

1. Clean the pycnometer and dry it. Find the mass ( $M_1$ ) of the pycnometer, brass cap and washer, accurate to 1 g.

2. Take about 200 to 400 g of oven-dried soil and put it in the pycnometer. Find the mass of the pycnometer plus soil etc. ( $M_2$ ).

3. Fill the pycnometer to half its height with distilled water and mix it thoroughly with glass rod. Add more water and stir it. Replace the screw top and fill the pycnometer flush with hole in the conical cap. Dry the pycnometer from outside, and find the mass ( $M_3$ ).

4. Empty the pycnometer ; clean it thoroughly and fill it with distilled water to the hole of the conical cap and find the mass ( $M_4$ ).

5. Repeat steps 2 to 4 for two more determinations of specific gravity.

**Tabulation of observations.** Table 3.9.

**TABLE 3.9 : DATA AND OBSERVATION SHEET FOR DETERMINATION OF SPECIFIC GRAVITY BY PYCNOMETER**

<i>Determination No.</i>	<i>1</i>	<i>2</i>	<i>3</i>
1. Pycnometer No.	5	5	5
2. Mass of Pycnometer ( $M_1$ ) g	652	652	652
3. Mass of Pyc. + soil ( $M_2$ ) g	908	950	929
4. Mass of Pyc. + soil + water ( $M_3$ ) g	1630	1656	1643
5. Mass of Pyc. + water ( $M_4$ ) g	1470	1470	1470
6. Specific Gravity	2.67	2.66	2.66
Av. Specific Gravity : 2.66			

**Calculations.** The Specific Gravity is calculated from Eq. 3.3 :

$$G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$

**EXPERIMENT 4 : DETERMINATION OF WATER CONTENT BY PYCNOMETER**

**Object and scope.** The object of the test is to determine the water content of a moist soil sample by pycnometer. For this determination, the specific gravity ( $G$ ) of soil solids must be known.

**Material and equipment.** (i) Pycnometer of about 900 ml capacity, with a conical brass cap screwed at its top, (ii) Balance sensitive to 1 g, (iii) Glass rod.

**Test Procedure**

1. Clean the pycnometer and dry it. Find the mass ( $M_1$ ) of the pycnometer, brass cap and the washer, accurate to 1 g.

2. Put about 200 g to 400 g of wet soil sample in the pycnometer and find its mass with its cap and washer ( $M_2$ ).

3. Fill the pycnometer to half its height and mix it thoroughly with the glass rod. Add more water and stir it. Replace the screw top and fill the pycnometer flush with the hole in the conical cap. Dry the pycnometer from outside and find its mass ( $M_3$ ).

4. Empty the pycnometer, clean it thoroughly, and fill it with clean water, to the hole of the conical cap and find its mass ( $M_4$ ).

Tabulation of observations. Table 3.10

**TABLE 3.10 : DATA AND OBSERVATION SHEET FOR WATER CONTENT DETERMINATION BY PYCNOMETER**

*Specific Gravity = 2.66*

1.	Mass of Pycnometer	( $M_1$ ) g	652
2.	Mass of Pyc. + moist soil	( $M_2$ ) g	891
3.	Mass of Pyc. + soil + water	( $M_3$ ) g	1608
4.	Mass of Pyc. + water	( $M_4$ ) g	1470
5.	Water content	( $w$ ) (%)	8

Calculations. Water content is calculated from Eq. 3.2 :

$$w = \left[ \frac{M_2 - M_1}{M_3 - M_4} \left( \frac{G - 1}{G} \right) - 1 \right] \times 100$$

### EXPERIMENT 5 : DETERMINATION OF DRY DENSITY AND DRY UNIT WEIGHT BY WATER DISPLACEMENT METHOD

**Object and scope.** The object of the test is to determine the dry density, dry unit weight and voids ratio of cohesive soil sample received from the field.

**Materials and equipment.** (i) A container about 30 cm diameter and 45 cm high, having an overflow outlet tube fitted in the upper half, (ii) A rubber tube with spring clip, fitted to the overflow outlet tube, (iii) Balance of 5 kg capacity, accurate to 1 g, (iv) Paraffin wax, of known specific gravity, (v) Container for water content determination, (vi) Thermostatically controlled oven, 105 °C to 110 °C, (vii) Glass beaker, 500 ml capacity, (viii) Cutting knife, (ix) Heater for melting wax and a brush.

#### Test Procedure

1. With the help of knife, trim the sample to a more or less regular shape, avoiding re-entrant angles. Find the mass of the wax coated sample ( $M_1$ ).

2. With the help of the brush, apply one coat of melted wax. When the coat is hardened, apply the second coat. Find the mass of the wax coated sample ( $M_2$ ).

3. Fill fresh water in the container to overflow through the tubing. When the overflow stops, clamp the clip of the tubing.

4. Immerse the wax coated sample slowly into the container, taking care that it sinks completely. Put the beaker below the rubber tubing and open the clip. Collect all the overflow water. Find the mass of water so collected, accurate to 1 g.

5. Take out the wax coated sample, dry it from outside and remove the paraffin wax skin. Cut the sample into two pieces and keep a representative sample for water content determination.

Tabulation of observations. The test observations are tabulated as illustrated in Table 3.11.

**TABLE 3.11. DATA AND OBSERVATION SHEET FOR DRY DENSITY DETERMINATION BY WATER DISPLACEMENT METHOD**

1.	Mass of the specimen	( $M_1$ ) (g)	768
2.	Mass of waxed specimen	( $M_2$ ) (g)	795
3.	Mass of wax coated	( $M_p = M_2 - M_1$ ) (g)	27
4.	Density of paraffin wax	( $G_p$ ) (g/ml)	0.908



5. Volume of wax coated	$V_p = \frac{M_p}{G_p}$ (ml)	30
6. Volume of water displaced	$(V_w)$ (ml)	424
7. Volume of specimen	$V = V_w - V_p$ (ml)	395
8. Bulk density	$\rho = \frac{M_t}{V}$ (g/cm <sup>3</sup> )	1.95
9. Bulk unit weight	$\gamma = 9.81 \rho$ (k N/m <sup>3</sup> )	19.13
10. Container No.		42
11. Mass of container and wet soil	(g)	50.35
12. Mass of container and dry soil	(g)	46.87
13. Mass of container	(g)	20.42
14. Mass of dry soil	(g)	26.45
15. Mass of water	(g)	3.48
16. Water content (w)	(Ratio)	0.13
17. Dry density of specimen	$\rho_d = \frac{\rho}{1 + w}$ (g/cm <sup>3</sup> )	1.73
18. Dry unit weight of specimen	$\gamma_d = 9.81 \rho_d$ (kN/m <sup>3</sup> )	16.97
19. Specific gravity of soil particles	(Assumed)	2.70
20. Voids Ratio	$e = \frac{G_{pw}}{\gamma_d} - 1$ (Ratio)	0.56

### EXPERIMENT 6 : DETERMINATION OF FIELD DENSITY AND DRY UNIT WEIGHT BY CORE CUTTER METHOD

**Object and scope.** The object of the test is to determine the dry density and dry unit weight of soil in-place by the core cutter.

**Materials and equipment.** (i) Cylindrical core cutter of steel, 130 mm long and 10 cm internal diameter, with a wall thickness of 3 mm, bevelled at one end, (ii) Steel dolly, 2.5 cm high and 10 cm internal diameter, with wall thickness 7.5 mm fitted with a lip to enable it to be fitted on top of the core cutter, (iii) Steel rammer, having mass of 9 kg (iv) Palette knife, (v) Steel rule, (vi) Spade or pickaxe or grafting tool, (vii) Straight edge, (viii) Balance accurate to 1 g, (ix) Container for water content determination.

#### Test Procedure

1. Measure the inside dimensions (accurate to 0.25 mm) of the core cutter and calculate its volume. Find the mass of the core cutter (without dolly), accurate to 1 g.

2. Expose the small area, about 30 cm square, to be tested and level it. Put the dolly on the top of the core cutter and drive the assembly into the soil with the help of the rammer until the top of the dolly protrudes about 1.5 cm above the surface.

3. Dig out the container from the surrounding soil, and allow some soil to project from the lower end of the cutter. With the help of the straight edge, trim flat the end of the cutter. Take out the dolly and also trim flat the other end of the cutter.

4. Find the mass of cutter full of soil.

5. Keep some representative specimen of soil for water content determination.

6. Repeat the test at two or three locations nearby and get the average dry density.

Tabulation of observations. Table 3.12.

**TABLE 3.12. DATA AND OBSERVATION SHEET FOR DRY DENSITY DETERMINATION BY CORE CUTTER METHOD**

<i>Determination No.</i>	<i>1</i>	<i>2</i>	<i>3</i>
1. Mass of core cutter + wet soil (g)	3027		
2. Mass of core cutter (g)	1058		
3. Mass of wet soil (g)	1969		
4. Volume of core cutter (ml)	1021		
5. Bulk density $\rho = \frac{(3)}{(4)}$ (g/cm <sup>3</sup> )	1.93		
6. Bulk unit weight $\gamma = 9.81 \rho$ (kN/m <sup>3</sup> )	18.92		
7. Container No.	8		
8. Mass of container + wet soil (g)	61.24		
9. Mass of container + dry soil (g)	56.36		
10. Mass of container (g)	20.76		
11. Mass of dry soil (g)	35.60		
12. Mass of water (g)	4.98		
13. Water content (Ratio)	0.14		
14. Dry density $\rho_d = \frac{\rho}{1 + w}$ (g/cm <sup>3</sup> )	1.69		
15. Dry unit weight $\gamma_d = \frac{\gamma}{1 + w}$ (kN/m <sup>3</sup> )	16.60		

**Reference to Indian Standard.** IS : 2720-1975/88 (Part XXIX) : Determination of dry density of soil in-place by the core-cutter method.

### EXPERIMENT 7 : DETERMINATION OF FIELD DENSITY AND DRY UNIT WEIGHT BY SAND REPLACEMENT METHOD

**Object and scope.** The object of the test is to determine the dry density of natural or compact soil, in-place, by the sand replacement method.

**Materials and equipment.** (i) Sand pouring cylinder of about 3 litre capacity, mounted above a pouring cone and separated by a shutter cover plate and a shutter, (ii) Cylindrical calibrating container, 10 cm internal diameter and 15 cm internal depth, fitted with flange approximately 5 cm wide and about 5 mm thick (iii) Glass plate, about 45 cm square and 1 cm thick, (iv) Metal tray with a central circular hole of diameter equal to the diameter of the pouring cone, (v) Tools for excavating hole, (vi) Balance accurate to 1 g, (vii) Container for water content determination, (viii) Clean, closely graded natural sand passing the 600-micron IS sieve and retained on the 300-micron IS sieve.

### Test Procedure

#### (A) *Determination of mass of sand filling the cone*

1. Fill the clean closely graded sand in the sand pouring cylinder upto a height 1 cm below the top. Determine the total initial mass of the cylinder plus sand ( $M_1$ ). This total initial mass should be maintained constant throughout the tests for which the calibration is used.

2. Allow the sand of volume equivalent to that of the excavated hole in the soil (or equal to that of the calibrating container), to run out of cylinder by opening the shutter. Close the shutter and place the cylinder on glass plate.

3. Open the shutter and allow the sand to run out. Close the valve when no further movement of sand is observed. Remove the cylinder carefully. Weigh the sand collected on the glass surface. Its mass ( $M_2$ ) will give the mass of sand filling the pouring cone. Repeat this step at least three times and take the mean mass ( $M_2$ ). Put the sand back into the cylinder, to have the same constant mass ( $M_1$ ).

#### (B) *Determination of bulk density of sand*

4. Determine the volume ( $V$ ) of the calibrating container by filling it with water full to the brim and finding the mass of water. This volume should be checked by calculating it from the measured internal dimensions of the container.

5. Place the sand pouring cylinder concentrically on the top of the calibrating container, after being filled to constant mass ( $M_1$ ). Open the shutter and permit the sand to run into the container. When no further movement of sand is seen, close the shutter. Remove the pouring cylinder and find its mass ( $M_3$ ) to nearest gram.

6. Repeat step (5) at least thrice and find the mean mass  $M_3$ . Put the sand into the sand pouring cylinder.

#### (C) *Determination of dry density of soil in-place*

7. Expose about 45 cm square area of the soil to be tested and trim it down to level surface. Keep the tray on the level surface and excavate a circular hole of approximately 10 cm diameter and 15 cm deep and collect all the excavated soil in the tray. Find the mass ( $M$ ) of the excavated soil.

8. Remove the tray, and place the sand pouring cylinder, so that the base of the cylinder concentrically covers the hole. The cylinder should have its constant mass  $M_1$ . Open the shutter and permit the sand to run into the hole. Close the shutter when no further movement of the sand is seen. Remove the cylinder and determine its mass ( $M_4$ ).

9. Keep a representative sample of the excavated soil for water content determination.

**Tabulation of observations.** The observations are tabulated as illustrated in Table 3.13.



TABLE 3.13. DATA AND OBSERVATION SHEET FOR DETERMINATION OF DRY DENSITY BY SAND REPLACEMENT METHOD

<b>(a) Determination of Mass of sand in the cone</b>			
1. Mass of sand (+ cylinder) before pouring	( $M_1$ ) (g)		10550
2. Mean mass of sand in cone	( $M_2$ ) (g)		445
<b>(b) Determination of bulk density of sand</b>			
3. Volume of calibrating container	( $V$ ) (ml)		1000
4. Mean mass of sand (+ cylinder) after pouring	( $M_3$ ) (g)		8655
5. Mass of sand filling calibrating container $M' = M_1 - M_3 - M_2$	(g)		1450
6. Bulk Density of sand	$\rho_s = \frac{(5)}{(3)}$ (g/cm <sup>3</sup> )		1.45
<b>(c) Bulk density and unit weight of soil</b>			
7. Mass of wet soil from the hole	( $M$ ) (g)		2234
8. Mass of sand (+ cylinder) after pouring in the hole	( $M_4$ ) (g)		8512
9. Mass of sand in the hole $M'' = M_1 - M_4 - M_2$	(g)		1593
10. Bulk density of soil $\rho = \frac{M}{M''} \times \rho_s$	(g/cm <sup>3</sup> )		2.03
11. Bulk unit weight of soil $\gamma = 9.81 \rho$	(kN/m <sup>3</sup> )		19.91
<b>(d) Water content determination</b>			
12. Container No.	(g)		11
13. Mass of container + wet soil	(g)		62.48
14. Mass of container + dry soil	(g)		57.76
15. Mass of container	(g)		21.43
16. Mass of dry soil	(g)		36.33
17. Mass of water	(g)		4.72
18. Water content ( $w$ )	(Ratio)		0.13
19. Dry density $\rho_d = \frac{\rho}{1 + w}$	(g/cm <sup>3</sup> )		1.8
20. Dry unit weight $\gamma_d = \frac{\gamma}{1 + w}$	(kN/m <sup>3</sup> )		17.62

Reference to Indian standard : IS : 2720-1974/88 (Part XXVIII): Determination of dry density by sand replacement method

#### EXPERIMENT 8 : DETERMINATION OF GRAIN SIZE DISTRIBUTION BY SIEVING

**Object and scope.** The object of this experiment is to determine grain-size distribution of coarse grained soil by sieving. The test covers both coarse sieve analysis (for gravel fraction) as well as fine sieve analysis (for sand fraction).

**Material and equipment.** (i) Balances accurate to 1 g and 0.1 g, (ii) Set of IS sieves : 100 mm, 75 mm, 22 mm, 10 mm, 4.75 mm, 2 mm, 1 mm, 600 micron, 425 micron, 300 micron, 212 micron, 150 micron and 75 micron size, (iii) Thermostatically controlled oven, (iv) Two or more large metal or plastic water tight trays, (v) Sieve brushes and a wire brush (vi) Mortar with a rubber covered pestle, (vii) Mechanical sieve-shaker and (viii) Riffler.

### Test Procedure

1. Using a riffler, take a representative sample of soil received from the field and dry it in the oven.

2. Weigh the required quantity of dried soil, keep it in a tray and soak it with water. Depending on the maximum size of material present in substantial quantities in the soil, the mass of soil sample taken for analysis may be as follows [IS : 2720 (Part IV)-1985].

<i>Maximum size of material present in substantial quantities (mm)</i>	<i>Mass to be taken for test (kg)</i>
75	60
40	25
25	13
19	6.5
12.5	3.5
10	1.5
6.5	0.75
4.75	0.40

3. Puddle the sample thoroughly in water and transfer the slurry to the 4.75 mm sieve which divides the gravel fraction from the sand fraction. Wash the slurry with jet of water. Collect the materials retained on 4.75 mm sieve and the material passing through it in separate containers. Keep the material retained on 4.75 mm sieve in the oven.

4. Wash the material passing through the 4.75 mm sieve through a 75-micron sieve so that silt and clay particles are separated from the sand fraction. Collect the material passing through 75-micron sieve and the material retained on it in separate containers, and keep them in the oven.

5. Sieve the dried material, retained on 4.75 mm sieve (step 3), through the following set of sieves : 100 mm, 19 mm, 10 mm and 4.75 mm by hand sieving. While sieving through each sieve, the sieve shall be agitated so that the sample rolls in irregular motion over the sieve. The material from the sieve may be rubbed, if necessary, with the rubber pestle in the mortar taking care to see that individual soil particles are not broken and re-sieved to make sure that only individual particles are retained. The mass of material retained on each sieve should be recorded.

6. Sieve the dried material, retained on 75-micron sieve (step 4), through the following set of sieves : 2 mm, 1 mm, 600 micron, 425 micron, 300 micron, 212 micron, 150 micron and 75 micron size. The set of sieves should be arranged one above the other and fitted to a mechanical sieve shaker such that the 2 mm sieve is at the top and the 75 micron sieve is at the bottom. A cover should be placed on the top of the 2 mm sieve, and a receiver should be placed below the 75 micron sieve. A minimum of 10 minutes sieving should be used. The soil fraction retained on each sieve should be carefully collected in containers and the mass of each fraction determined and recorded.

*Alternatively,* the material retained on 75 micron sieve (step 4), may not be dried, but be washed through a nest of sieves specified in step 6, nested in order of their fineness with the finest (75 micron) at the bottom. Washing should be continued until the water passing through each sieve is substantially clean. The fraction retained on each sieve should be emptied carefully without loss of material in separate containers and oven-dried. The oven-dried fraction should be weighed separate and their mass should be recorded.

7. The material passing 75 micron sieve (step 4) may be used for sedimentation analysis. (See Experiments 9 or 10).

**Tabulation of observations.** The test observations and results are recorded as illustrated in Table 3.14.

**Calculations.** The percentage of soil retained on each sieve is calculated on the basis of total mass of soil sample taken and from these results the percent passing through each of the sieve is calculated, as illustrated in Table 3.14.

**TABLE 3.14. DATA AND OBSERVATION SHEET FOR SIEVE ANALYSIS**

Sample No. 108

Mass retained on 4.75 mm sieve : 167 g

Mass of dry soil sample : 1000 g

Mass passing through 75 micron size : 77 g

S.N.	IS Sieve	Particle size D (mm)	Mass retained (g)	% retained	Cumulative % retained	Cumulative % finer (N)
1.	100 mm	100 mm	—	—	—	100
2.	75 mm	75 mm	—	—	—	100
3.	19 mm	19 mm	33	3.3	3.3	96.7
4.	10 mm	10 mm	49	4.9	8.2	91.8
5.	4.75 mm	4.75 mm	85	8.5	16.7	83.3
6.	2 mm	2 mm	140	14.0	30.7	69.3
7.	1 mm	1 mm	160	16.0	46.7	53.3
8.	600 micron	0.6 mm	142	14.2	60.9	39.1
9.	425 micron	0.425 mm	118	11.8	72.7	27.3
10.	300 micron	0.300 mm	82	8.2	80.9	19.1
11.	212 micron	0.212 mm	56	5.6	86.5	13.5
12.	150 micron	0.150 mm	35	3.5	90.0	10.0
13.	75 micron	0.075 mm	23	2.3	92.3	7.7

**Note 1. Dry Sieve Analysis.** If the soil sample contains little or no fines (passing 75 micron sieve), dry sieve analysis may be carried out. The gravel fraction and sand fraction are first separated by dry sieving through 4.75 mm sieve. The material retained on 4.75 mm size is further sieved through the following set of sieves: 100 mm, 75 mm, 19 mm, 10 mm and 4.75 mm sieves. The material passing 4.75 mm sieve is sieved through the following set of sieves: 2 mm, 1 mm, 600 micron, 425 micron, 300 micron, 212 micron, 150 micron and 75 micron sizes.

**Note 2.** The permissible maximum mass of sample on the 200 mm diameter sieves should be as follows :

IS Sieve Designation	Maximum mass of sample (g)
600 micron	160
200 micron	55
75 micron	25

**Note 3.** If the soil contains greater amount of fines (finer than 75 micron size), the dried soil fraction passing through 4.75 mm sieve (step 6) should be soaked in water containing two grams of sodium hexametaphosphate (or one gram of sodium hydroxide and one gram of sodium carbonate) per litre of water. The soaked specimen should then be washed thoroughly over the nest of sieves specified in step 6.



## EXPERIMENT 9 : DETERMINATION OF GRAIN SIZE DISTRIBUTION BY HYDROMETER

**Object and scope.** The object of this experiment is to determine the distribution of particle size, finer than 75 micron sieve, by sedimentation analysis, using a density hydrometer, and then to plot the grain size distribution curve.

**Materials and equipment.** (i) Density hydrometer conforming to IS : 3104-1965, (ii) Two glass measuring cylinders of 1000 ml capacity with ground glass or rubber stoppers about 7 cm diameter and 33 cm high marked at 1000 ml volume, (iii) Thermometer to cover the range 0 to 50 °C, accurate to 0.50 °C, (iv) Water bath or constant temperature room (optional), (v) Stirring apparatus, (vi) 75 micron sieve, (vii) Balance accurate to 0.01 g, (viii) Stop watch, (ix) Wash bottles containing distilled water, (x) Glass rod, about 15 to 20 cm long and 4 to 5 mm in diameter, (xi) Reagents : Hydrogen peroxide, Hydrochloric acid N solution and Sodium hexametaphosphate, (xii) Conical flask of 1000 ml capacity, (xiii) Buchner or Hirsch funnel, (xiv) Filter flask, (xv) Measuring cylinder of 100 ml capacity, (xvi) Filter paper and blue litmus paper.

### Test Procedure

#### (A) Calibration of hydrometer

1. *Determination of volume of the hydrometer bulb ( $V_b$ ).* Pour about 800 ml of water in the 1000 ml measuring cylinder and note the reading at the water level. Immerse the hydrometer in water and note the water reading. The difference between the two readings is recorded as the volume of the hydrometer bulb plus the volume of that part of the stem which is submerged. For practical purposes, the error due to the inclusion of this stem volume may be neglected.

*Alternatively,* weigh the hydrometer to the nearest 0.2 g. This mass in grams is recorded as the volume of the hydrometer in ml. This includes the volume of the bulb plus the volume of the stem. For practical purposes the error due to the inclusion of the stem may be neglected.

2. In order to find the area of cross-section ( $A$ ) of the measuring cylinder in which the hydrometer is to be used, measure the distance, in cm, between two graduations of the cylinder. The cross-sectional area ( $A$ ) is then equal to the volume included between the two graduations divided by the distance between them.

3. Measure the distance ( $h$ ) from the neck to the bottom of the bulb, and record it as the height of the bulb.

4. With the help of an accurate scale, measure the height  $H$  between the neck of the hydrometer to each of the other major calibration marks ( $R_h$ ).

5. Calculate the effective depth ( $H_e$ ) corresponding to each of the major calibration marks (or hydrometer readings,  $R_h$ ) by the following expression :

$$H_e = H + \frac{1}{2} \left( h - \frac{V_b}{A} \right)$$

The readings may be recorded as illustrated in Table 3.3.

6. Draw a calibration curve between  $H_e$  and  $R_h$  (Fig. 3.7) which may be used for finding the effective depth ( $H_e$ ) corresponding to hydrometer readings ( $R_h$ ) obtained during the test.

7. *Meniscus correction.* Insert the hydrometer in the measuring cylinder containing about 700 ml of water. Take the readings of the hydrometer at the top and bottom of the meniscus. The difference between two readings is taken as the meniscus correction ( $C_m$ ) which is a constant for a hydrometer. During the actual sedimentation test, the readings should be taken at the bottom of the meniscus but since the soil suspension is opaque, readings are taken at the top of meniscus. It is clear from Fig. 3.6 (a) that readings decrease in the upward direction. Thus, the observed hydrometer readings is always less than the true one. Hence the meniscus correction is always positive.

**(B) Pre-treatment of soil**

1. Weigh accurately (to 0.01 g) 50 to 100 g of oven-dried soil sample ( $M_d$ ) passing the 2 mm IS sieve (50 g for clay soil and 100 g if it is a sandy soil). If the percentage of soluble salts is more than one per cent, the soil should be washed with water before further treatment, taking care to see that the soil particles are not lost.

2. Add 150 ml of hydrogen peroxide to the soil sample placed in a wide mouth conical flask and stir it gently for few minutes with a glass rod. Cover the flask with glass and level it to stand overnight.

3. Next morning, the mixture in the conical flask is gently heated in an evaporating dish, stirring the contents periodically. Reduce the volume to about 50 ml by boiling. With very organic soils additional peroxide may be required to complete the oxidation.

4. If the soil contains insoluble calcium compounds, add about 50 ml of hydrochloric acid to the cooled mixture of soil obtained in step 3. The solution is stirred with a glass rod for a few minutes and allowed to stand for one hour or for longer periods, if necessary. The solution will have an acid reaction to litmus.

5. Filter the mixture and wash it with warm water until the filtrate shows no acid reaction to litmus. Transfer the damp soil on the filter paper and funnel to the evaporating dish using a jet of distilled water. Place the dish and its contents to the oven. Take the mass ( $M_f$ ) of the oven-dried soil remaining after pre-treatment and find the loss of mass due to pre-treatment.

**Note.** In case of soils containing no calcium compounds or soluble salts having a low organic content (less than 20%) the pre-treatment prescribed above may be omitted and the dispersing agent is added direct to the soil taken for analysis.

**(C) Dispersion of soil**

1. To the oven-dried soil in the evaporating dish (step 5 above), add 100 ml of sodium hexametaphosphate solution and warm the mixture gently for about 10 minutes. Transfer the mixture to the cup of the mechanical mixer using a jet of distilled water, and stir it well for about 15 minutes. The sodium hexametaphosphate solution is prepared by dissolving 33 g of sodium hexametaphosphate and seven grams of sodium carbonate in distilled water to make one litre of solution. This solution is unstable and should be freshly prepared approximately once in a month.

2. Transfer the soil suspension to the 75 micron IS sieve placed on a receiver and wash the soil on this sieve using jet of distilled water from a wash bottle. The amount of distilled water used during this operation may be about 500 ml.

3. Transfer the soil suspension passing the 75 micron IS sieve to the 1000 ml measuring cylinder, and add more distilled water to make the volume to exactly 1000 ml in the cylinder.

4. Collect the material retained on 75 micron sieve and put it in the oven for drying. Determine the dry mass of soil retained on 75 micron sieve.

**(D) Sedimentation test with hydrometer**

1. Insert a rubber bung or any other suitable cover on the top of the 1000 ml measuring cylinder containing the soil suspension (obtained in step 3 above) and shake it vigorously end over end. Stop shaking and allow it to stand. Immediately, start the stop watch, and remove the top cover from the cylinder.

2. Immerse the hydrometer gently to a depth slightly below its floating position and then allow it to float freely. Take the hydrometer readings after periods of 1/2, 1, 2 and 4 minutes. Take out the hydrometer, rinse it with distilled water and allow it to stand in a jar containing distilled water at the same temperature as that of the test cylinder.

3. The hydrometer is re-inserted in the suspension and readings are taken after periods of 8, 15 and 30 minutes ; 1, 2 and 4 hours after shaking. The hydrometer should be removed, rinsed and placed in the distilled water after each reading. After the end of 4 hours, readings should be taken once or twice within 24 hours.

4. **Composite correction.** In order to determine the composite correction, put 100 ml of dispersing agent solution in another 1000 ml measuring cylinder and make it to 1000 ml by adding distilled water. The cylinder should be maintained at the same temperature as that of the test cylinder containing soil specimen. Insert the hydrometer in this comparison cylinder containing distilled water and the dispersing agent and take the reading corresponding to the *top* of the meniscus. The *negative* of the hydrometer reading so obtained gives the composite correction ( $C$ ). The composite correction is found before the start of the test, and also at every time intervals of 30 minutes, 1 hour, 2 hours and 4 hours after the beginning of the test, and afterwards, just after each hydrometer reading is taken in test cylinder.

5. The temperature of the suspension should be observed and recorded once during the first 15 minutes and then after every subsequent reading.

**Tabulation of observations.** The test observations and results are recorded as illustrated in Table 3.15. The observations for the calibration of the hydrometer have been recorded in Table 3.3.

**Particle size distribution curve.** The results of the above table are plotted to get a particle size distribution curve with percentage finer  $N$  as the ordinate and the particle diameter ( $D$ ) on logarithmic scale as abscissa.

#### Calculations.

(1) The loss in mass in pre-treatment of the soil in percentage is calculated from the following expression :

$$P = \left( 1 - \frac{M_b}{M_d} \right) 100$$

where  $P$  = loss in mass in percentage

$M_d$  = mass of dry soil sample taken from the soil passing 2 mm sieve

$M_b$  = mass of the soil after pre-treatment.

(2) The diameter of the particles in suspension at any sampling time  $t$  is calculated from Eq. 3.9 :

$$D = 10^{-5} F \sqrt{\frac{H_e}{t}}, \text{ where factor } F \text{ is taken either from Table 3.2 or from Fig. 3.4.}$$

(3) The percentage finer  $N'$  based on the mass  $M_d$  is calculated from Eq. 3.19

$$N' = \frac{100 G}{M_d (G - 1)} R.$$

(4) The percentage finer  $N$  based on total mass of dry soil sample ( $M$ ) is obtained from the

relation : 
$$N = N' \times \frac{M'}{M}$$

where  $M'$  = cumulative mass passing 2 mm sieve

(out of which the soil weighing  $M_d$  was taken for the hydrometer analysis).

**TABLE 3.15. DATA AND OBSERVATION SHEET FOR HYDROMETER ANALYSIS**

1. Sample No. A--102
2. Mass of dry soil sample ( $M'$ ) = 500 g
3. Mass of fraction passing 2 mm sieve ( $M'$ ) = 500 g
4. Mass of dry sample taken from minus 2 mm sieve ( $M_d$ ) = 50 g
5. Specific gravity of soil particles fo minus 75 micron :  $G = 2.67$
6. Hydrometer No. 12 : Sedimentation jar No. 36 ; Meniscus correction :  $C_m = + 0.5$ .



Date	Time	Elapsed time $t$ (min)	Hydro-meter Reading $R_h'$	Temp $^{\circ}\text{C}$	Correc-tion $C$	$R_h = R_h' + C_m$	Eff. depth $H_e$ (cm)	Factor $F$	Particle size $D$ (mm)	$R = R_h' + C$	% finer ( $N'$ ) based on $M_d$	% finer ( $N$ ) based on whole $N = N' \times \frac{M'}{M}$
12-4-76	08-27	1/2	20.25	26.5	- 0.5	20.75	12.2	1254	0.061	19.75	63	58.0
		1	19.00	26.5	- 0.5	19.50	12.7	1254	0.045	18.50	59	54.3
		2	17.50	26.5	- 0.5	18.00	13.3	1254	0.033	17.00	54.4	50.0
		4	15.50	26.5	- 0.5	16.00	14.0	1254	0.023	15.00	48	44.1
		8	15.50	26.5	- 0.5	16.00	14.0	1254	0.0165	15.00	48	44.1
		15	11.50	27.0	- 0.5	12.00	15.5	1252	0.0125	11.00	35	32.2
	08-57	30	9.50	27.5	- 0.5	10.00	16.3	1245	0.0092	9.00	29	26.7
	09-30	63	7.25	29	0	7.75	17.1	1224	0.0062	7.25	23	21.2
	10-20	113	6.00	30	+ 0.5	6.50	17.6	1210	0.0047	6.50	21	19.3
	12-27	240	4.00	31.5	+ 1	4.50	18.3	1190	0.0033	5.00	16	14.7
	17-00	513	2.25	31.5	+ 2	2.75	19.0	1190	0.0023	3.25	10.4	9.6
	07-55	1408	2.25	29	0	2.75	19.0	1224	0.0015	2.25	6.5	5.8

(5) The uniformity coefficient  $C_u$  and coefficient of curvature  $C_c$  are calculated respectively from Eqs. 3.21 and 3.22.

**Note 1.** If the soil sample does not contain calcium compounds or soluble salts, pre-treatment should be omitted.

**Note 2.** Sodium hexametaphosphate has been found to be ineffective when dealing with certain highly flocculated soils. In such cases dispersion may be carried out by adding N-sodium hydroxide solution at the rate of 4 ml per 10 g of soil.

**Note 3.** Asymmetrical heating of the suspension cause convection currents which affect the sedimentation process. The suspension should therefore, be kept out of direct sunlight and away from any local source of heat. Evaporation should be retarded by keeping a cover on the measuring cylinder between the readings.

**Note 4.** The specific gravity  $G$  used in the above expression should be determined for the fraction of the sample passing 75 micron sieve.

## EXPERIMENT 10 : DETERMINATION OF GRAIN SIZE DISTRIBUTION BY PIPETTE

**Object and scope.** The object of the experiment is to determine the distribution of particle size, finer than 75 micron sieve, by sedimentation analysis, using a sampling pipette, and then to plot the grain size distribution curve.

**Materials and equipment.** (i) A sampling pipette fitted with a pressure and suction inlet having a capacity of approximately 10 ml fitted on a sliding panel, (ii) Glass sedimentation tubes (2 Nos.), 50 mm diameter and approx. 350 mm long marked at 500 ml volume, with rubber bungs to fit, (iii) Weighing bottles with round stopper, (iv) Constant temperature bath, capable of being maintained at  $27^{\circ} \pm 0.1^{\circ}\text{C}$ , (v) Stirring apparatus, (vi) 75 micron sieve, (vii) Balance to weigh upto 0.001 g, (viii) Stop watch, (ix) Desiccator, (x) Evaporating dish, (xi) Conical beaker of capacity 650 ml or 1 litre with a cover glass to fit, (xii) Funnel and filter flask, (xiii) Measuring cylinder of 1000 ml capacity, (xiv) Pipette of 25 ml capacity, (xv) Glass filter funnel of 10 cm diameter, (xvi) Glass rod, (xvii) Filter papers, Blue litmus paper, (xviii) Thermometer to cover the range  $0^{\circ} - 50^{\circ}\text{C}$ , (xix) Reagents : Hydrogen Peroxide, Hydrochloric acid N solution and Sodium hexametaphosphate, (xx) Wash bottles containing distilled water.

**Test Procedure :** (A) **Pre-treatment of soil**

(1) Weigh accurately to 0.001 g, a quantity  $M_d$  of oven-dried soil sample passing the 2 mm IS sieve. The quantity  $M_d$  varies from 30 g for sandy soil to about 12 g with clay soil.

(2) Place the soil in a 650 ml conical beaker and add 55 ml of distilled water to it. Boil the suspension gently until the volume is reduced to 40 ml. After cooling, add 75 ml of hydrogen peroxide and allow to stand overnight.

(3) Next morning, gently heat the suspension, avoiding frothing over and agitating the contents frequently. As soon as vigorous frothing has subsided and when there is no further reaction by the addition to fresh hydrogen peroxide, the volume is reduced to about 30 ml by boiling.

(4) If the soil contains insoluble calcium compounds, add about 10 ml of hydrochloric acid to the cooled mixture. The solution is stirred with a glass rod for a few minutes and allowed to stand for one hour or longer periods if necessary. The treatment should be continued till the solution gives acid reaction to litmus. More acid may be required for soil containing considerable amount of calcium salts.

(5) Filter the mixture using the Buchner funnel and wash it with warm water until the filtrate shows no acid reaction to litmus. Transfer the damp soil on the filter paper and funnel to the evaporating dish using a jet of distilled water. Place the dish and its contents to the oven. Find the mass  $M_b$  of the oven-dried soil remaining after pre-treatment, accurate to 0.001 g.

**Note.** In case of soils containing no calcium compounds or soluble salts, and having a low organic content (less than 2%) the pre-treatment prescribed above may be omitted and the dispersing agent is added direct to the soil taken for analysis.

#### **(B) Dispersion of soil**

1. To the oven-dried soil in the evaporating dish (step 5 above) add 25 ml of sodium hexametaphosphate solution, together with about 25 ml of distilled water, and the soil is brought into suspension by stirring with a glass rod. Warm the mixture gently for about 10 minutes and transfer it to the cup of the mechanical mixer using a jet of distilled water and stir it well for about 15 minutes. The sodium hexametaphosphate solution is prepared by dissolving 33 g of sodium hexametaphosphate and 7 grams of sodium carbonate in distilled water to make one litre solution.

2. Transfer the soil suspension to the 75 micron IS sieve placed on a receiver and wash the soil on this sieve using jet of distilled water from a wash bottle. The amount of distilled water used during this operation should not exceed 150 ml.

3. Transfer the soil suspension passing 75 micron sieve to the 500 ml sedimentation tube, and add more distilled water to make the volume to exactly 200 ml. Keep the tube in the constant temperature bath.

4. Collect the material retained on 75 micron sieve and put it in the oven for drying. Determine the dried mass accurate to 0.001 gram.

#### **(C) Sedimentation test with pipette**

1. **Calibration of sampling pipette.** Clean the sampling pipette and dry it. Immerse the nozzle in distilled water and close the cock of the bulb. Open the three way cock to connect the pipette to the suction outlet. With the help of rubber tubing attached to the suction outlet, suck up the water to rise above the three way cock. Close three way cock and remove the pipette from water. Remove the surplus water in the small bulb by connecting it to the water outlet through the three way cock. Now the water contained in the pipette and the three way cock is discharged into a glass weighing bottle. Determine the mass of water in grams, which will give the volume of the pipette.

2. The sedimentation tube containing the soil suspension, and with rubber bung inserted at its top, is shaken end over end. The tube is then replaced in the bath, and stop watch is simultaneously started. The rubber bung is carefully removed.

3. (a) Fill the upper bulb with distilled water after closing the cock below it. To collect the sample at any instant  $t$  after the start of the test, lower the pipette into the suspension until its end



is 10 cm below the top of suspension, just 15 seconds before the sample is due to be taken. Connect the three way cock to the suction tube and suck the sample slowly till the sample comes just above the three way cock. This operation should take about 10 seconds.

(b) Take out the pipette from the suspension. Drain out the excess sample collected in the small bulb by connecting it to the water outlet through the three ways cock. Open the cock below the bulb to allow the distilled water to run from the bulb to the water outlet until no solution remains in the system.

(c) To collect the soil suspension contained in the pipette, insert glass weighing bottle below the nozzle of the pipette and open the three way stop cock so that the contents are drained into the bottle. Any suspension left on the inner walls of the pipette is washed into the weighing bottle by allowing distilled water from the upper bulb to run into the pipette through the intermediate small bulb.

4. Samples are drawn, as explained in step 3 above, after 0.5, 2, 4, 8, 15 and 30 minutes, and 1, 2, 4, 8, 16 and 24 hours, reckoned from the commencement of the test.

5. The weighing bottles, into which soil suspension are collected at various time intervals, are placed into the oven and the samples are evaporated to dryness. After cooling in a desiccator the weighing bottles, the mass of contents are found to the nearest 0.001 g.

**Tabulation of observations.** The test observations and results are recorded as illustrated in Table 3.16.

**TABLE 3.16. DATA AND OBSERVATION SHEET FOR PIPETTE ANALYSIS**

Soil No. B/31

Dry mass of soil taken : 20 g

Specific Gravity : 2.70

Mass retained on 75 micron sieve: 3.96 g

Volume of suspension : 500 ml

% Finer than 75 micron : 80%

Mass of Dispersing agent in the suspension : 1 g

Date and Time	Elapsed time (min.)	Temp. (°C)	Factor F	D (mm)	Bottle No.	Bottle + Dry mass	Mass of Bottle	Dry mass of soil	$M_D$	N%
12.1.73 11.32	$\frac{1}{2}$	20	1347	0.0604	2	22.942	22.610	0.332	0.0332	78.0
	2	20	1347	0.0300	4	22.224	21.914	0.310	0.0310	72.5
	4	20	1347	0.0213	5	22.681	22.397	0.284	0.0284	66.0
	8	20	1347	0.0151	6	22.679	22.418	0.261	0.0261	60.3
	15	20	1347	0.0110	7	22.804	25.571	0.233	0.0233	53.3
12.02	30	20	1347	0.0078	8	22.987	22.773	0.214	0.0214	48.5
	60	20	1347	0.0055	9	22.610	22.414	0.196	0.0196	44.0
	120	21	1331	0.0039	10	22.315	22.142	0.173	0.0173	38.3
	240	21	1331	0.0034	11	22.416	22.269	0.147	0.0147	33.7
13.1.73 11.32	1440	20	1347	0.0012	12	22.570	22.483	0.087	0.0087	16.8

**Particle size distribution curve.** The results of Table 3.16 are plotted to get a particle size distribution curve with  $N$  as ordinate and  $D$ , on logarithmic scale, as abscissa.

**Calculations.** 1. The loss of mass if pre-treatment of the soil in percentage is calculated from the following expression :

$$P = \left( 1 - \frac{M_b}{M_d} \right) 100$$

where  $P$  = loss in mass in percentage

$M_d$  = mass of dry soil sample taken from the soil passing 2 mm sieve

$M_b$  = mass of the soil after pre-treatment

2. The diameter of the particle in suspension at any sampling time  $t$  is calculated from Eq. 3.9 :



$$D = 10^{-5} F \sqrt{\frac{H_d}{l}} = 10^{-5} F \sqrt{\frac{10}{l}}$$

where factor  $F$  is taken either from Table 3.2 or from Fig. 3.4:

3. The percentage finer  $N$  based on  $M_d$  is calculated from Eq. 3.13 :

$$N = \frac{M_D - \frac{m}{V}}{M_d/V} \times 100$$

where  $V$  = Total volume of suspension = 500 ml

$m$  = Mass of dispersing agent present in the total suspension of volume  $V = \frac{33 + 7}{1000} \times 25 = 1$  g

$M_d$  = Mass of dry soil sample taken from the soil passing 2 mm sieve

$M_D$  = Dry mass of the sample in the weighing bottle per ml of the suspension

$$= \frac{\text{Dry mass of sample in the weighing bottle}}{\text{Volume } (V_p) \text{ of the pipette}}$$

4. The percentage finer  $N$  based on total mass of dry soil sample ( $W$ ) is obtained from the relation:

$$N = N' \times \frac{M'}{M}$$

where  $M'$  = Cumulative mass passing 2 mm sieve (out of which the mass  $M_d$  was taken for the hydrometer analysis).

5. The uniformity coefficient  $C_u$  and the coefficient of curvature  $C_c$  are calculated from Eqs. 3.21 and 3.22 respectively.

## EXPERIMENT 11 : DETERMINATION OF LIQUID LIMIT OF SOIL

**Object and scope.** The object of the test is to determine the liquid limit of the soil sample using Casagrande type mechanical liquid limit apparatus.

**Material and equipment.** (i) Mechanical liquid limit device (Casagrande type) consisting of a brass cup and carriage, mounted on base Micarta Number 221 A, (ii) Grooving tool 'a' (Casagrande or B.S. tool) and grooving tool 'b' (ASTM tool), (iii) Porcelain evaporating dish, about 12 cm in diameter or marble plate 30 cm square, (iv) Flexible spatula, with blade about 8 cm along and 2 cm wide, (v) Balance to weigh to 0.01 g, (vi) Airtight containers to determine water content, (vii) Thermostatically controlled oven to maintain temperature between 105° to 110° C, (viii) Wash bottle containing distilled water, (ix) 425 micron sieve, (x) Desiccator.

### Test Procedure

1. By means of the gauge on the handle of the grooving tool and the adjustment plate, adjust the height through which the cup is lifted and dropped so that the point on the cup which comes in contact with the base falls through exactly one centimetre when the handle is rotated by one revolution. When the adjustment is complete, secure the adjustment plate by tightening its screws.

2. Take about 120 g of the specimen, passing through the 425 micron sieve, and mix it thoroughly with distilled water in the evaporating dish or on the marble plate so that uniform paste is formed. Leave the soil for sufficient time so that water may permeate throughout the soil mass. In the case of fat clays, this maturing time may be upto 24 hours. For an average soil, thorough mixing for about 15 to 30 minutes may be sufficient. The amount of water to be added depends on the type of soil, and is a matter of experience.

3. Take a portion of the paste with the spatula and place it in the centre of the cup so that it is almost half filled. Level off the top of the wet soil symmetrically with the spatula, so that it is parallel to the rubber base and the maximum depth of the soil is 1 cm.

4. With the help of grooving tool 'a', the paste in the cup is divided along the cup diameter (through the centre line of the cam follower), by holding the tool normal to the surface of the cup and drawing it firmly across. Thus, a V-shaped gap, 2 mm wide at the bottom and 11 mm at the top and 8 mm deep will be formed. However, in the case of sandy soils tool 'a' does not form a neat groove and hence tool 'b' is used.

5. Turn the handle of the apparatus at the rate of 2 revolutions per seconds, until the two parts of the soil come in contact with the bottom of the groove along a distance of 10 mm. Record the number of blows required to cause the groove close for approximate length of 10 mm.

6. Collect a representative slice of soil by moving the spatula widthwise from one edge to the other edge of the soil cake at right angles to the groove, including the portion of the groove in which the soil flowed together, and put it in an airtight container. Its water content is later determined by method of Expt. 1.

7. Remove the remaining soil from the cup and mix it with the soil left earlier on the marble plate (or evaporating dish). Change the consistency of the mix by either adding more water or leaving the soil paste to dry, as the case may be, and repeat steps 3, 4, 5 and 6. Note the number of revolutions to close the groove and keep the soil for water content determination. These operations are repeated for 3 or 4 more additional trials. The soil paste in these operations should be of such consistency that number of revolutions or drops to close the groove is  $25 \pm 10$ . The test should always proceed from the dryer (more blows) to the wetter (less blows) condition of the soil.

**Tabulation of observations.** The observations are tabulated as illustrated in Table 3.17.

**TABLE 3.17. DATA AND OBSERVATION SHEET FOR LIQUID LIMIT DETERMINATION**

Soil No. C-108

Date

Determination No.	1	2	3	4
Number of blows	34	23	18	12
Container number	130	123	128	132
Mass of container + wet soil (g)	38.86	46.63	60.36	43.43
Mass of container + dry soil (g)	34.91	39.59	49.02	37.22
Mass of water (g)	3.95	7.04	11.34	6.21
Mass of container (g)	26.08	25.30	26.95	66.06
Mass of oven dry soil (g)	8.83	14.29	22.07	11.16
Water content (%)	44.6	49.4	51.4	55.6
Liquid limit (from graph) = 48.5%. Flow index (from graph) = 227				

**Calculations and results.** Plot the flow curve (Fig. 3.11) with water content as the ordinate and log of number of blows as abscissa. The water content corresponding to 25 blows is taken as the liquid limit of the soil.

From the graph, find the difference between the water contents for the blows differing by one log cycle. To do this, extend the flow curve at either ends so as to intersect the ordinates corresponding to 10 and 100 blows, and note the numerical difference in water contents at 10 and 100 blows.

**Note 1.** Some soils tend to slide on the surface of the cup instead of 'flowing'. In such a case, the test should be repeated. If slipping still occurs, the test is not applicable, and a note to this effect should be made.

**Note 2.** It has been observed that heating a soil alters its liquid limit. Generally, liquid limit of natural soil is required. Drying may alter soil by causing the particles to subdivide or agglomerate by driving off absorbed water which is not completely regained on rewetting, or by effecting a chemical change in any organic matter in the soil. Hence the soil should not be oven-dried before the commencement of the test.

**Note 3.** Distilled water should be used in order to minimise the possibility of ion exchange between the soil and any impurities in the water.

**Note 4.** It is better if the test is performed in a humid room. Humid atmosphere minimizes surface drying of the soil while it is being tested.

**Reference to Indian Standard.** IS : 2720 (Part V)-1985. Determination of liquid and plastic limits.

## EXPERIMENT 12 : DETERMINATION OF PLASTIC LIMIT OF SOIL

**Object and scope.** The object of the test is to determine the plastic limit of soil sample and then to calculate plasticity index, toughness index, liquidity index and consistency index of the soil.

**Materials and equipment.** The materials and equipment of experiment 11 plus a rod of 3 mm diameter.

### Test Procedure

1. Take about 20 g of air dried soil from the thoroughly mixed portion of the material passing 420 micron IS sieve. Mix it on the marble plate with sufficient distilled water to make it plastic enough to be shaped into a ball. Leave the plastic soil mass for some time to mature. In some fat clays, the plastic soil mass may be left to stand for 24 hours to allow water to permeate throughout the soil mass.

2. Take about 8 g of the plastic soil, make a ball of it, and roll it on the marble (or glass) plate with the hand with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. When the diameter of the thread has decreased to 3 mm, the specimen is kneaded together and rolled out again. Continue the process until thread just crumbles at 3 mm diameter.

3. Collect the crumbled soil thread in the airtight container and keep it for water content determination. The test is repeated *twice* more. Thus three readings are obtained for the determination.

4. Also, determine the natural water content of the soil sample obtained from the field.

**Tabulation of observations.** The observations are tabulated as illustrated in Table 3.18.

**TABLE 3.18 : DATA AND OBSERVATION SHEET FOR PLASTIC LIMIT DETERMINATION**

Soil No. C-108

Determination Number	1	2	3
Container Number	33	19	22
Mass of container + wet soil (g)	31.29	30.39	30.87
Mass of container + oven dry soil (g)	29.75	28.75	29.27
Mass of water (g)	1.54	1.64	1.60
Mass of container (g)	24.01	22.79	23.42
Mass of dry soil (g)	5.74	5.96	5.85
Water content (%)	26.8	27.5	27.3
Plastic limit = 27.2%		Natural water content of field soil = 32.8%	

### Calculations

1. Plastic limit = 27.2
2. Plasticity index =  $48.5 - 27.2 = 21.3$  (since  $w_L = 48.5\%$ )
3. Liquidity index  $I_L = \frac{w - w_p}{I_p} = \frac{32.8 - 27.2}{21.3} = 0.263$
4. Consistency index  $I_c = \frac{w_L - w}{I_p} = \frac{48.5 - 32.8}{21.3} = 0.737$
5. Flow Index  $I_f = 22.7$  (from Experiment 11)
6. Toughness Index  $I_T = \frac{I_p}{I_f} = \frac{21.3}{22.7} = 0.94$

**Summary of results.** Given in Table 3.19.



TABLE 3.19

<i>Liquid Limit</i> <i>w<sub>L</sub></i>	<i>Flow Index</i> <i>I<sub>f</sub></i>	<i>Plastic Limit</i> <i>w<sub>P</sub></i>	<i>Plasticity Index</i> <i>I<sub>P</sub></i>	<i>Liquidity Index</i> <i>I<sub>L</sub></i>	<i>Consistency Index</i> <i>I<sub>C</sub></i>	<i>Toughness Index</i> <i>I<sub>T</sub></i>
48.5	22.7	27.2	21.3	0.263	0.734	0.94

**Note 1.** Undue pressure or oblique rolling, which might result in mechanical breaking of the soil thread, should not be used. The soil thread must crumble due to its decrease in water content only. The rate of rolling should be between 80 and 90 strokes per minute counting a stroke as one complete motion of the hand forward and back to the starting position again.

**Note 2.** In the case of sandy soils plastic limit should be determined first. When the plastic limit cannot be determined, the plasticity index should be reported as NP (non-plastic).

**Note 3.** When the plastic limit is equal to or greater than the liquid limit, the plasticity index should be reported as zero.

**Reference to Indian Standard.** IS : 2720 (Part V)-1985. Determination of liquid and plastic limits.

### EXPERIMENT 13 : DETERMINATION OF SHRINKAGE FACTORS OF SOIL

**Object and scope.** The object of the experiment is to determine shrinkage limit, shrinkage ratio and volumetric shrinkage.

**Materials and equipment.** (i) Evaporating dish (2 Nos.) of procelain, about 12 cm in diameter with flat bottom, (ii) Shrinkage dish, of non-corroding metal, having flat bottom and 45 mm in diameter and 15 mm in height internally (3 Nos.), (iii) Glass cup, 50 to 55 mm in diameter and 25 mm in height, the top rim of which is ground smooth and level, (iv) Glass plates, (2 Nos.), each 75 × 75 mm. One plate should be of plain glass and the other should have three metal prongs, (v) Spatula, (vi) Straight edge, (vii) 425 micron IS sieve, (viii) Balances, sensitive to 0.1 g and 0.01 g, (ix) Oven, thermostatically controlled (105° – 110°C), (x) Mercury, (xi) Desiccator, (xii) Wash bottle containing distilled water.

#### Test Procedure (for Remoulded sample)

1. **Preparation of soil paste.** Take about 100 g of soil sample from a thoroughly mixed portion of the material passing 425 micron IS sieve.

2. Place about 30 g of the above sample in evaporating dish and mix it thoroughly with distilled water. Water added should be sufficient to fill the voids in the soil completely and make the soil pasty enough to be readily worked into the shrinkage dish without entrapping air bubbles. In the case of plastic soils, the water content of the paste may exceed its liquid limit by as much as 10 percent, while for friable soil the amount of water required to obtain the desired consistency may be equal to or slightly greater than the liquid limit.

3. **Determination of mass and volume of the shrinkage dish.** Clean a shrinkage dish and determine its mass accurate to 0.1 g. To determine its volume, place the shrinkage dish in the evaporating dish and fill it to overflowing the mercury. Remove the excess mercury by pressing the plain glass plate firmly on its top, taking care that no air is entrapped. Wipe off, carefully, any mercury which may be adhering to the outside of the shrinkage dish. Carefully transfer the mercury of the shrinkage dish to the other evaporating dish, and then determine the mass of mercury, accurate to 0.1 g. The mass of mercury divided by its density would give the volume of the shrinkage dish, which is also the volume of wet soil pat.

4. **Filling the shrinkage dish soil pat.** Coat the inside of the shrinkage dish with a thin layer of silicon grease or vaseline. In the centre of dish, place the soil paste, about one-third of the volume of the dish, with the help of spatula. Tap the dish gently on a firm surface, cushioned with layers of blotting paper or rubber sheet and allow the paste to flow towards the edges. Place another equal instalment of the paste in the dish and make it flow toward the edges by tapping. Tapping should be

continued till the paste is compacted and all the entrapped air is brought to the surface. Repeat the process till the dish is completely filled and the excess soil overflows. Strike off the excess soil paste with a straight edge. Wipe off the soil adhering to the outside of the dish.

5. *Determination of wet and dry mass of soil pat.* Weigh immediately the shrinkage dish plus the wet soil pat, accurate to 0.1 g. Keep the shrinkage dish open to air until the colour of pat turns from dark to light. Keep the shrinkage dish in the oven and thus dry the pat to constant mass at 105° to 110° C. Cool the dish in a desiccator and weigh immediately.

6. *Determination of volume of dry soil pat.* To determine the volume of the dry soil pat, keep the glass cup in the evaporating dish. Fill the cup to overflowing with mercury. Remove the excess mercury by pressing the glass plate with the three prongs firmly over the top of the cup. Transfer the cup carefully to another evaporating dish, carefully wiping off any mercury which may be adhering to the outside of the cup. Place the oven-dried soil pat on the surface of mercury in the cup and carefully force the pat into the mercury by pressing it by the same glass plate containing three prongs. Press the plate firmly on the top of the cup. Collect carefully the displaced mercury and find its mass to an accuracy of 0.01 g. The volume of the dry soil pat is then determined by dividing this mass by the density of mercury.

**Alternative Procedure (for undisturbed soil)**

(i) Undisturbed soil sample obtained from the field is trimmed approximately 45 mm in diameter and 15 mm in height. The edges of the pat are rounded off so that no air is entrapped during mercury displacement.

(ii) Keep the pat in a suitable container and air dry it for some time so that its colour changes dark to light. Keep the container in the oven and dry it at 105° to 110° C. Take out the pat and smoothen the edges by sand paper.

(iii) Place the pat again in the oven for some time and dry it to constant mass. Cool the even dried part in the desiccator, and then determine its dry mass.

(iv) Determine the volume of the dry soil pat, as explained in step (5) above.

Tabulation of observations. Table 3.20 and 3.21.

**TABLE 3.20. DATA AND OBSERVATION SHEET FOR SHRINKAGE FACTORS (REMOULDED SAMPLE)**

<i>Determination Number</i>	<i>1</i>	<i>2</i>	<i>3</i>
<b>(a) Water content of wet soil pat</b>			
1. Shrinkage dish No.	1	2	3
2. Mass of shrinkage dish	23.0	22.2	21.7
3. Mass of shrinkage dish + wet soil pat (g)	63.5	61.3	62.0
4. Mass of shrinkage dish + dry soil pat (g)	52.6	50.9	51.3
5. Mass of dry soil pat ( $M_d$ ) (g)	29.6	28.7	29.6
6. Mass of water (g)	10.9	10.4	10.7
7. Water content of soil pat ( $w$ ) (Ratio)	36.8	36.3	36.2
<b>(b) Volume of wet soil pat</b>			
8. Evaporating dish No.	1	1	1
9. Mass of mercury filling shrinkage dish + mass of evaporating dish (g)	350.3	345.4	348.5
10. Mass of evaporating dish (g)	76.2	76.2	76.2
11. Mass of mercury filling shrinkage dish (g)	274.1	269.2	272.3
12. Vol. of wet soil pat $V = \frac{(11)}{13.6}$ (cm <sup>3</sup> )	20.2	19.8	20.2

Determination Number	1	2	3
<b>(c) Volume of dry soil pat</b>			
13. Evaporating dish No.	1	1	1
14. Mass of mercury displaced by dry soil pat + mass of evaporating dish (g)	304.3	300.6	302.7
15. Mass of evaporating dish (g)	76.2	76.2	76.2
16. Mass of mercury displaced by dry soil pat (g)	228.1	224.4	226.5
17. Vol. of dry soil pat $V_d = \frac{(16)}{13.6}$ (cm <sup>3</sup> )	16.8	16.5	16.7
<b>(d) Calculations</b>			
18. Shrinkage limit $w_s = \left[ w - \frac{V - V_d}{M_d} \right] 100$ (%)	25.3	24.8	24.4
19. Shrinkage ratio $SR = \frac{M_d}{V_d}$	1.76	1.74	1.77
20. Volumetric shrinkage $VS = (w - w_s)SR$	20.2%	20.0%	20.9%

TABLE 3.21. DATA AND OBSERVATION SHEET FOR SHRINKAGE LIMIT (UNDISTURBED SAMPLE)

Determination No.	1	2	3
1. Dish No.	1	2	3
2. Mass of dish + dry soil pat (g)	52.1	50.7	50.6
3. Mass of dish (g)	23.1	22.2	21.7
4. Mass of dry soil pat ( $M_d$ ) (g)	29.0	28.5	28.9
5. Evaporating dish No.	1	1	1
6. Mass of mercury displaced by dry pat + Mass of evaporating (g)	308.6	306.2	310.2
7. Mass of evaporating dish (g)	76.2	76.2	76.2
8. Mass of mercury displaced by dry soil pat (g)	232.4	230.0	234.0
9. Volume of dry soil pat $V_d = \frac{(8)}{13.6}$ (cm <sup>3</sup> )	17.1	16.9	17.2
10. Specific gravity of soil ( $G$ )	2.70	2.70	2.70
11. Shrinkage limit $w_s = \left( \frac{V_d}{M_d} - \frac{1}{G} \right) 100$ (%)	22.0	22.3	22.5

**Calculations.** The various factors are calculated as under :

$$1. \text{ Shrinkage limit (remoulded sample) } w_s = \left[ w - \frac{V - V_d}{M_d} \right] \times 100 \quad (\text{Since } \rho_w = 1 \text{ g/cm}^3)$$

$$2. \text{ Shrinkage Ratio : } SR = \frac{M_d}{V_d \rho_w} = \frac{M_d}{V_d}$$

$$3. \text{ Volumetric Shrinkage : } VS = (w - w_s) SR$$

$$4. \text{ Shrinkage Limit (undisturbed sample) : } w_s = \left( \frac{V_d}{M_d} - \frac{1}{G} \right) \times 100$$

- Results**
1. Shrinkage Limit (Remoulded Sample) : 24.8%
  2. Shrinkage Ratio : 1.76
  3. Volumetric Shrinkage : 20.4%
  4. Shrinkage Limit (Undisturbed Sample) : 22.3%

**Reference to Indian Standard.** IS : 2720-1972/78 (Part VI) : Determination of Shrinkage Factors.



### PROBLEMS

1. During a sedimentation test for grain size analysis, the corrected hydrometer reading in a 1000 ml uniform soil suspension at the instant of commencement of sedimentation is 1.030. After 30 minutes, the corrected hydrometer reading is observed as 1.015, and the corresponding effective depth is 10 cm. If the specific gravity of soil solids is 2.68 and viscosity of water is 0.01 Poise, find (i) the total mass of soil solids placed in the 1000 ml suspension, (ii) the effective diameter corresponding to the 30 minutes reading, and (iii) the percentage finer than this diameter.

Ans. (i) 47.9 g, (ii) 0.0078 mm, (iii) 50%

2. To determine the in-situ density of a compacted embankment, a small hole is dug into the embankment and the excavated soil is weighed. Uniformly graded dry sand is then poured into the hole from a weighed sand pouring cylinder fitted with pouring cone at its bottom. The cylinder after filling the hole is weighed. The mass of sand filling the pouring cone is determined afterwards. The density of sand used in the cylinder is determined by filling it in a 1000 cm<sup>3</sup> calibrating can whose empty mass is 944 g. If the water content of the embankment soil is 8% and other test data are as given below, determine the in-situ dry density :

(i) Mass of excavated soil = 925 g

(ii) Mass of cylinder + sand before test = 5332 g

(iii) Mass of cylinder + sand after filling the hole = 4152 g

(iv) Mass of sand filling the pouring cone = 432 g

(v) Mass of calibrating can + sand = 2483 g. [Ans. 1.76 g/cm<sup>3</sup> ; 17.27 kN/m<sup>3</sup>]

3. The Atterberg limits of a soil sample are  $w_L = 50\%$ ,  $w_P = 30\%$  and  $w_S = 15\%$ . If the specimen of this soil shrinks from a volume of 10 cm<sup>3</sup> at liquid limit to 5.94 cm<sup>3</sup> when it is oven-dried, calculate (i) shrinkage ratio, and (ii) specific gravity of the soil solids.

[Ans. (i) 1.96, (ii) 2.78]

4. A saturated soil sample has a volume of 20 cm<sup>3</sup> at its liquid limit. Given  $w_S = 42\%$ ,  $w_L = 17\%$  and  $G = 2.74$ , find the minimum volume which the soil can attain.

[Ans. 14.2 cm<sup>3</sup>]

5. The oven-dry mass of pat of clay is 10.8 g and the mass of mercury displaced on immersion is 84.2 g. Taking the specific gravity of solids as 2.72, determine the shrinkage limit and shrinkage ratio.

[Ans. 20.4% ; 1.745]

6. A clay sample has a voids ratio of 0.53 in dry state. What will be its shrinkage limit if  $G = 2.70$  ?

[Ans. 19.6%]

7. Define liquid limit, liquidity index and consistency index. Determine the value of the liquid limit of a soil from the following test data :

Number of blows	Water content (%)
38	16
34	17
20	20
12	22

[Ans. 18.7%]

8. An oven-dry sample of volume 220 cm<sup>3</sup> has a mass of 400 g. Determine its voids ratio and shrinkage limit, if the specific gravity of soil particles is assumed as 2.76. If the sample is allowed to swell and get fully saturated on contact with water, what will be the water content which will fully saturate the sample and also cause an increase in volume equal to 10% of the original dry volume ?

[Ans.  $e = 0.535$  ;  $w_S = 19.4\%$  ;  $w = 24.2\%$ ]

# Classification of Soils

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## 4.1. GENERAL

The purpose of soil classification is to arrange various types of soils into groups according to their engineering or agricultural properties and various other characteristics. Soil possessing similar characteristics can be placed in the same group. Soil survey and soil classification are carried out by several agencies for different purposes. For example, the agriculture departments undertake soil investigations from the point of view of the suitability, or otherwise, of the soil for crops and its fertility. However, from engineering point of view, the classification may be done with the objective of finding the suitability of the soil for construction of dams, highways or foundations, etc. For general engineering purposes, soils may be classified by the following systems :

1. Particle size classification,
2. Textural classification,
3. Highway Research Board (HRB) classification,
4. Unified soil classification and IS classification system.

## 4.2. PARTICLE SIZE CLASSIFICATION

In this system, soils are arranged according to the grain size. Terms such as gravel, sand, silt and clay are used to indicate grain sizes. These terms are used only as *designation of particles size*, and do not signify the naturally occurring soil types, which are mixtures of particles of different sizes and exhibit definite characteristics. It is preferable to use the word '*silt size*' and '*clay size*' in place of simply 'silt' or 'clay' in this system. There are various grain size classifications in use, but the more commonly used systems are :

(i) U.S. Bureau of Soil and Public Road Administration (PRA) System of United States, (ii) International soil classification, proposed at the International Soil Congress at Washington, D.C. in 1927, and (iii) The M.I.T. classification proposed by Prof. Gilboy of Massachusetts Institute of Technology as a simplification of the Bureau of Soils Classification, and (iv) Indian Standard Classification (IS : 1948-1970) based on the M.I.T. system. These four systems are shown in Fig. 4.1.

## 4.3. TEXTURAL CLASSIFICATION

Soils occurring in nature are composed of different percentage of sand, silt and clay size particles. Soil classification of composite soils exclusively based on the particle size

	0.005 mm	0.05	0.10	0.25	0.50	1.0	2.0 mm
Clay (Size)	Silt (Size)	V. F.	Fine	Medium	Coarse	Fine Gravel	Gravel
		Sand					

(a) U. S. Bureau of soils and PRA classification

	0.0002	0.0006	0.002	0.006	0.02	0.05	0.1	0.2	0.5	1.0	2.0 mm
Ultra Clay	F	C	F	C	F	C	F	M	C	V.C.	Gravel
(Colloids)	Clay		Silt		MO (Majla)		Sand				

(b) International Classification

	0.0002	0.006	0.02	0.06	0.2	0.6	2.0 mm
Clay (Size)	Fine	Med.	Coarse	Fine	Med.	Coarse	Gravel
(Colloids)	Silt (Size)			Sand			

(c) M.I.T. Classification

	0.002 mm	0.075	0.425	2	4.75	20	80	300
Clay (Size)	Silt (Size)	Fine	Med.	Coarse	Fine	Coarse	Cobble	Boulder
		Sand			Gravel			

(d) I.S. Classification (IS : 1498-1970)

FIG. 4.1. GRAIN-SIZE CLASSIFICATION SCALES.

distribution is known as textural classification. Probably the best known of these textural classifications is the triangular classification of U.S. Public Roads Administration, shown in Fig. 4.2. The classification is based on the percentages of sand, silt and clay sizes making up the soil. Such a classification is more suitable for describing coarse grained soils rather than clay soils whose properties are less dependent on the particle size distribution.



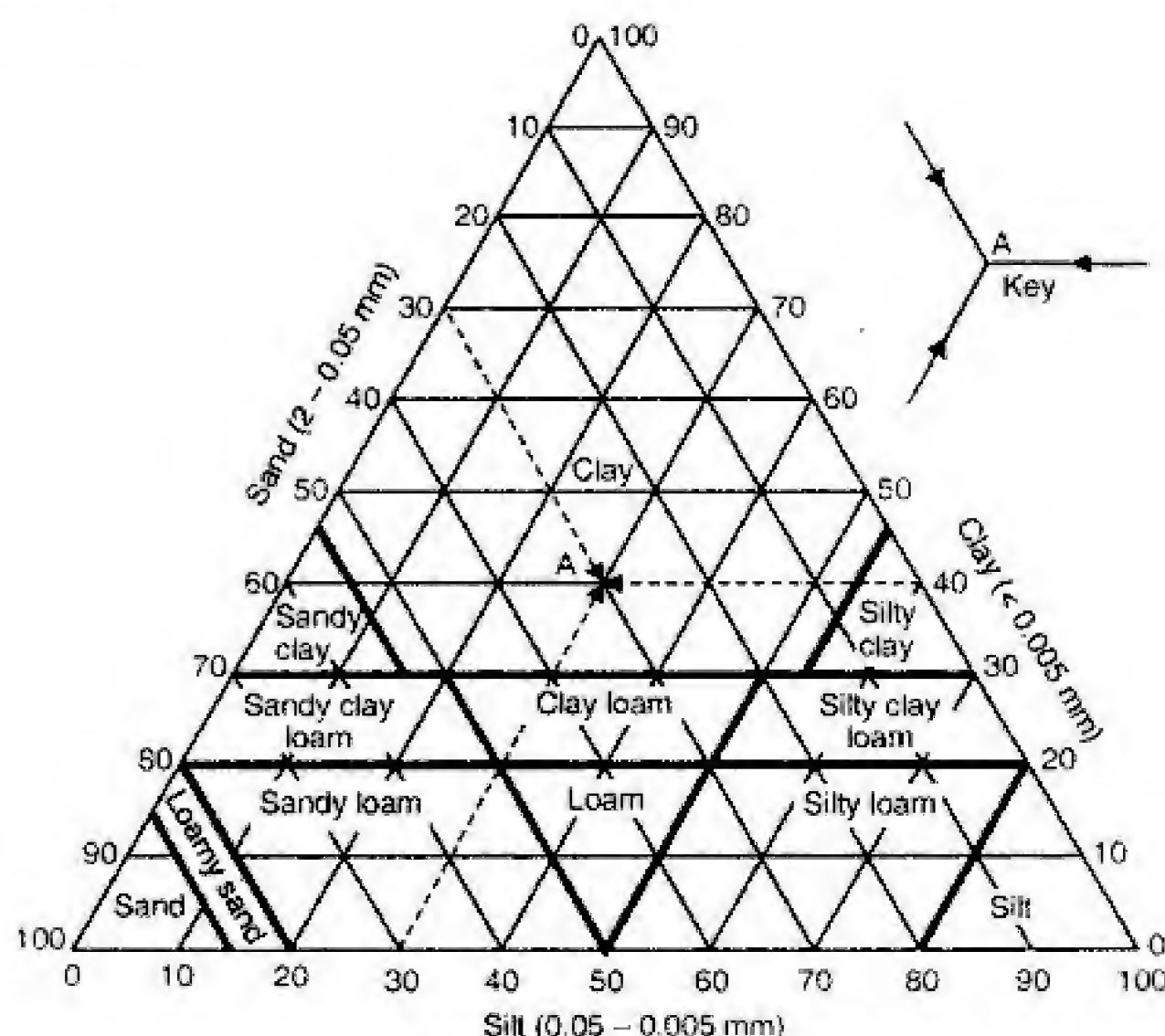


FIG. 4.2. TEXTURAL CLASSIFICATION CHART  
(ADAPTED FROM U.S. PUBLIC ROADS ADMINISTRATION)

To use the chart, for the given percentages of the three constituents forming a soil, lines are drawn parallel to the three sides of the equilateral triangle, as shown by arrows in the 'key' of Fig. 4.2. For example, if a soil is composed of 30 per cent sand, 30 per cent silt sizes and 40 per cent clay sizes, the three lines so drawn intersect at the point *A* situated in the sector designated as 'clay'. Such a soil will be termed as 'clay'.

#### 4.4 HIGHWAY RESEARCH BOARD (HRB) CLASSIFICATION

The Highway Research Board (HRB) classification system, also known as Public Road Administration (PRA) classification system, is based on both the particle-size composition as well as the plasticity characteristics. The system is mostly used for pavement construction. Soils are divided into 7 primary groups, designated as A - 1, A - 2...A - 7 as shown in Table 4.1. Group A - 1 is divided into two sub-groups and group A - 2 into four sub-groups. A characteristic *group index* is used to describe the performance of the soils when used for pavement construction. Group index is *not* used to place a soil in particular group; it is actually a means of rating the value of a soil as a sub-grade material within its own group. The higher the value of the index, the poorer is the quality of the material.

The group index of a soil depends upon (i) the amount of material passing the 75-micron IS sieve, (ii) the liquid limit, and (iii) the Plastic limit, and is given by the following equation :

$$\text{Group index} = 0.2a + 0.005ac + 0.01bd \quad \dots(4.1)$$

where  $a$  = that portion of percentage passing 75 micron sieve greater than 35 and not exceeding 75 expressed as a whole number (0 to 40)  
 $b$  = that portion of percentage passing 75 micron sieve greater than 15 and not exceeding 55 expressed as a whole number (0 to 40)  
 $c$  = that portion of the numerical liquid limit greater than 40 and not exceeding 60 expressed as positive whole number (0 to 20)  
and  $d$  = that portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20).

TABLE 4.1. HRB-CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Description	Granular materials (35% or less passing 75 micron IS sieve)							Silt clay materials (more than 35% passing 75 micron IS sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percent passing 2.0 mm IS sieve	50 max										
425 micron sieve	30 max	50 max	51 min								
75 micron sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 425 micron sieve											
Liquid Limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		NP	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Group Index	Zero					4 max		8 max	12 max	16 max	20 max
Usual type of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

**Classification procedure.** With required test data available, proceed from left to right on the chart (Table 4.1) and correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification. The plasticity index of A-7-5 sub-group is equal to or less than liquid limit minus 30. Plasticity index of A-7-6 sub-group is greater than liquid limit minus 30.

The group index should be rounded off to the nearest whole number and placed in parenthesis, such as A-2-2(6) or A-6(6). See example 4.3.

**Classification of Indian Black Cotton soils.** The divisions of A-7 group on the basis of the demarcation line ( $I_p = w_L - 30$ ) into A-7-5 and A-7-6 sub-groups does

not appear to divide the Indian Black Cotton soils into two distinct groups having maximum value of group index as 20 only. Based on investigations carried out at the Central Road Research Laboratory, New Delhi (1953), a classification of black cotton soil into narrow sub-groups (Table 4.2) has been suggested extending the maximum value of group index from 20 to 50. Accordingly the factors for the calculation of group index (Eq. 4.1) have the following maximum values :  $a = 65$  ;  $b = 65$  ;  $c = 45$  ; and  $d = 34$ .

TABLE 4.2. EXTENSION OF HRB CLASSIFICATION FOR INDIAN BLACK COTTON SOILS

Group	A-7	A-7 a	A-7 b	A-7 c
Per cent passing 75 micron sieve	upto 75	55-95	80-95	85-100
Characteristics of fraction passing 425 micron sieve				
Liquid limit	below 55	below 65	above 65	above 65
Plasticity index	below 25	below 42	below 42	above 42
Group index	20 max	20 - 30	30 - 40	40 - 50
Usual type of significant constituent material	Clayey soils			
General rating as sub-grade	Fair to Poor	Poor		

#### 4.5. UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

Originally developed by Casagrande (1940), the Unified Soil Classification System (USCS) was used for air field construction during World War II. It was later (1952) modified slightly by the Bureau of Reclamation and Corps of Engineers of USA, to make it applicable to other constructions like foundations, earth dams, earth canals, earth slopes etc. The system has also been adopted by American Society of Testing Materials (ASTM) and later by Bureau of Indian Standards (1970). According to the USCS, the coarse grained soils are classified on the basis of their grain size distribution while the fine-grained soils, whose behaviour is controlled by plasticity, are classified on the basis of their plasticity.

Various soils are classified into four major groups : (i) Coarse grained, (ii) Fine grained, (iii) Organic soils, and (iv) Peat. Table 4.3 gives the group symbols consisting of a prefix and a suffix. Thus, there are in all 15 groups of soils – 8 groups of coarse grained soils, 6 groups of fine grained soils (including organic soils), and 1 group for peat. The soils are first classified into two categories : (i) coarse grained soils, and (ii) fine grained soils.

TABLE 4.3. PREFIXES AND SUFFIXES OF USCS

Soil Type	Prefix	Sub Group	Suffix
(1) Gravel	G	(i) Well graded	W
(2) Sand	S	(ii) Poorly graded	P
(3) Silt	M	(iii) Silty	M
(4) Clay	C	(iv) Clayey	C
(5) Organic	O	(v) $w_L < 50\%$	L
(6) Peat	Pt	(vi) $w_L > 50$	H

1. **Coarse grained soils.** If more than 50% of the soil is retained on No. 200 US sieve (0.075 mm), it is designated as coarse grained soil. A coarse grained soil is





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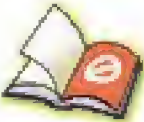
TABLE 4.5. CHART FOR ENGINEERING USE OF SOILS (WAGNER, 1957)

Typical Names of Soil Groups	Groups Symbols	Important Properties				Relative Desirability for Various Uses									
		Permeability when Compacted	Shearing Strength when Compacted and Saturated	Compressibility when Compacted and Saturated	Workability as a Construction Material	Rolled Earth Dams			Canal Sections		Foundations		Roadways		
						Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage not Important	Fills	Frost Heave not Possible	Surfacing
Well graded gravels, gravel-sand mixtures, little or no fines	GW	pervious	excellent	negligible	excellent	-	-	1	1	-	-	1	1	1	3
Poorly graded gravels, gravel sand mixtures, little or no fines	GP	very pervious	good	negligible	good	-	-	2	2	-	-	3	3	3	-
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	semipervious to impervious	good	negligible	good	2	4	-	4	4	1	4	4	9	5
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	impervious	good to fair	very low	good	1	1	-	3	1	2	6	5	5	1
Well graded sand, gravelly sand, little or no fines	SW	pervious	excellent	negligible	excellent	-	-	3 if gravelly	6	-	-	2	2	2	4
Poorly graded sands, gravelly sands, little or no fines	SP	pervious	good	very low	fair	-	-	4 if gravelly	7 if gravelly	-	-	5	6	4	-
Silty sands poorly graded sand-silt mixtures	SM	semipervious to impervious	good	low	fair	4	5	-	8 if gravelly	5 erosion critical	3	7	8	10	6
Clayey sands, poorly graded sand-clay mixtures	SC	impervious	fair	low	good	3	2	-	5	2	4	8	7	6	2





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TABLE 4.7. IS SOIL CLASSIFICATION (IS : 1498-1970)  
(INCLUDING FIELD IDENTIFICATION AND DESCRIPTION)

Major Divisions		Group Symbol	Typical Names	Field identification procedures (excluding particles larger than 80 mm and basing fractions on estimated weights)	Information required for describing soils
1	2	3	4	5	6
COARSE-GRAINED SOILS More than half of material is larger than 75-micron sieve size The 75-micron sieve size is about the smallest particle visible to the naked eye	GRAVELS More than half of coarse fraction is larger than 4.75 mm sieve size	3	4	5	6
	SANDS More than half of coarse fraction is smaller than 4.75 mm sieve size (For visual classification, the 5 mm size may be used as equivalent to the 4.75 mm sieve size)	3	4	5	6
	SANDS More than half of coarse fraction is smaller than 4.75 mm sieve size (For visual classification, the 5 mm size may be used as equivalent to the 4.75 mm sieve size)	3	4	5	6
	SANDS More than half of coarse fraction is smaller than 4.75 mm sieve size (For visual classification, the 5 mm size may be used as equivalent to the 4.75 mm sieve size)	3	4	5	6
	SANDS More than half of coarse fraction is smaller than 4.75 mm sieve size (For visual classification, the 5 mm size may be used as equivalent to the 4.75 mm sieve size)	3	4	5	6



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2. Liquid limit	30%	52%
3. Flow index	11	6
4. Natural water content	32%	40%

Find which soil is (a) more plastic, (b) better foundation material on remoulding (c) better shear strength as a function of water content, (d) better shear strength at plastic limit. Classify the soil as per ISCS. Do these soils have organic matter ?

**Solution :**

(a) Plasticity index  $I_p$  for soil A =  $30 - 16 = 24$

Plasticity index  $I_p$  for soils B =  $52 - 19 = 33$

Since plasticity index of soil B is greater, soil B is more plastic.

(b) Consistency index  $I_c$  for soil A =  $\frac{w_L - w}{I_p} = \frac{30 - 32}{24} = -0.083$

Consistency index  $I_c$  for soil B =  $\frac{w_L - w}{I_p} = \frac{52 - 40}{31} = 0.387$

The consistency index for soil A is negative. Hence it will turn into slurry when remoulded. Hence soil A is not suitable for foundations. However, soil B will be suitable.

(c) Flow index  $I_f$  for soil A = 11; Flow index  $I_f$  for soil B = 6

Since the flow index of soil B is lesser than of soil A, soil B has better shear strength as a function of water content.

(d) Toughness index  $I_T$  for soil A =  $I_p / I_f = 24 / 11 = 2.18$

Toughness index  $I_T$  for soil B =  $33 / 6 = 5.5$

Since the toughness index for soil B is greater than that of A, soil B has better shear strength at plastic limit.

**Classification of the soil as per ISCS:** When  $I_p$  and  $w_L$  are marked on the plasticity chart, soils A and B fall in the zones of CL and CH respectively. Thus soil A is inorganic clay for low plasticity while soil B is inorganic clay of high plasticity. Hence these soils do not have organic matter.

**Example 4.3.** Laboratory tests on a soil sample obtained from a highway project site reveal that 56 percent of the soil passes 200 No. US sieve and the liquid and plastic limits of the soil are 36 percent and 23 percent respectively. Determine the group index of the soil and classify it as per HRB/PRA classification system.

**Solution :** Refer Table 4.1.

As more than 35% of the soil passes through 200 No. US sieve (or 75 micron IS sieve) the soil falls in the group between A-4 to A-7. Since the liquid limit is less than 40, the soil falls either in A-4 group or in A-6 group. Further, since the plastic limit is more than 21, it falls in A-6 group.

$$\text{Group index} = 0.2a + 0.005ac + 0.01bd \quad \dots(\text{Eq.4.1})$$

where  $a$  = that portion of the percent passing 200 No. sieve (75 micron) greater than 35 and not exceeding 75 =  $56 - 35 = 21$ .



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TABLE 4.10. CHARACTERISTICS PERTINENT TO ROADS AND AIRFIELDS

Soil group	Value as sub-grade when not subject to frost Action	Value as sub-base when not subject to frost Action	Value as base when not subject to frost Action	Potential frost Action	Compressibility and Expansion	Drainage Characteristics	Compaction Equipment	Unit Dry density $\text{g/cm}^3$	CBR values	Subgrade modulus $(K) \text{ kg/cm}^2$
1	2	3	4	5	6	7	8	9	10	11
GW	Excellent	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tyre roller, steel-wheeled roller	2.00-2.24	40-80	8.3-13.84
GP	Good to excellent	Good	Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tyre roller, close control of moisture	1.76-2.24	30-60	8.3-13.84
GM	d	Good to excellent	Good	Fair to good	Very slight	Fair to poor	Rubber-tyred roller, sheepfoot roller, close control of moisture	2.00-2.32	40-80	8.3-13.84
	u	Good	Fair	Slight to medium	Slight	Poor to practically impervious	Rubber-tyred roller, sheepfoot roller	1.84-2.16	20-30	5.53-8.3
GC	Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tyred roller, sheepfoot roller	2.08-2.32	20-40	5.53-8.3
SW	Good	Fair to good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tyred roller	1.76-2.08	20-40	5.53-11.07
SP	Fair to good	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tyred roller	1.68-2.16	10-40	4.65-11.07
SM	d	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tyred roller, sheepfoot roller, close control of moisture	1.92-2.16	15-40	4.65-11.07
	u	Fair	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tyred roller, sheepfoot roller	1.60-2.08	10-20	2.77-8.3
SC	Poor to Fair	Poor	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tyred roller, sheepfoot roller	1.60-2.16	5-20	2.77-8.3
ML, MI	Poor to Fair	Not suitable	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tyred roller, sheepfoot roller, close control of moisture	1.44-2.08	15 or less	2.77-5.3





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we shall briefly describe the various electrical forces which participate in the building of a soil structure.

## 5.2. SOLID PARTICLES IN SOILS

The particles of coarse-grained soils are composed of primary minerals (*i.e.*, they are the same as existing in present-day rocks). These particles are sometimes termed as *bulky* particles, and can be thought of rough-edged shapes approaching spheres. These particles do not possess the property of plasticity and cohesion and their behaviour is governed primarily by gravitational forces or mass energy rather than colloidal forces.

Many investigations have shown (Grim, 1959) that fine-grained soils are composed predominantly of crystalline minerals and the amorphous materials that may be present (such as allophane) have little, if any, effect on soil behaviour. These minerals, which have low surface activities and do not contribute appreciable plasticity or cohesion are referred to as *non-clay minerals*. The crystalline minerals whose surface activity is such that they develop cohesion and plasticity are called *clay minerals*. About 15 minerals are classed as clay minerals and these belong to four main groups : kaolin, montmorillonite, illite and playgorskite. Chemically, the clay minerals are silicates of aluminium and/or iron and magnesium. Some of them also contain alkalies and/or alkaline earths, as essential components. Most of the clay minerals have sheet or layered structures. Some of the clay minerals have elongate tubular or fibrous structures. Clays can be considered as essentially made up of extremely small particles, each one of which is either a book of sheet-like units or a bundle of tubes or fibres. Individual soils or clays may contain more than one kind of book-like units or a mixture of books and bundles of tubes or fibres (Grim, 1959). Clay particles behave like *colloids*. A colloid is a particle whose specific surface (surface area per unit mass or volume) is so high that its behaviour is controlled by surface energy rather than mass energy. The smaller is any given shaped particle, the larger is the surface area per unit volume. If we consider a cube of sides 1 cm long, the ratio of surface area to the volume is 6 per centimetre. Suppose the cube is sub-divided into smaller cubes whose sides are  $1 \mu (= 10^{-3} \text{ mm})$  in length, there will be  $10^{12}$  such cubes in a total volume of  $1 \text{ cm}^3$  and a total surface area of  $6 \times 10^4 \text{ sq. cm.}$  increasing the ratio of surface area to volume by ten thousand times. Since the clay particles are plate, needle or rod shaped, they have still higher specific surfaces than a cube of equal volume. A montmorillonite platelet  $0.1 \mu \times 0.002 \mu$  has specific surface/volume equal to  $1000/\mu$ , which is five times that of an equivolume cube. The upper size limit of a colloid has arbitrarily been set at approximately 0.2 to  $1 \mu$ . Nearly, all clay particles are colloidal even though the maximum particles dimension of several of clay minerals (such as kaolinite, dickite, attapulgite etc.) is greater than  $1 \mu$ .

## 5.3. ATOMIC AND MOLECULAR BONDS

The nature of surface bonding forces is not completely understood. Possibly, one of the simplest general classification system for these bonding forces is : (1) Electrostatic or Primary Valence Bond, (2) Hydrogen Bond, and (3) Secondary Valence Bond. Lambe (1953) recognises the hydrogen bond as a special type of secondary valence bond. Atoms



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ions or exchangeable ions, since they can be replaced. If there were no thermal activity possessed by these ions, and if there were no attraction exerted on them by other ions and colloids, these counter ions would all swarm to the surface of the particles to neutralise the surface charge of the particle. Thus, their positions are compromises between the particle charge which pulls them in and their thermal activities plus the attraction by other bodies, which keeps them away. The counter ions thus constitute the *diffuse double layer* (Fig. 5.6) of the colloid; the surface charge of the colloid is the other layer of the double layer. The force fields that develop between the charged soil particles, the surrounding water, and the associated ions have a controlling influence on the soil properties which can be varied within wide limits by changing such factors as the types of ions, and their concentration, temperature, and the nature and amount of pore fluid.

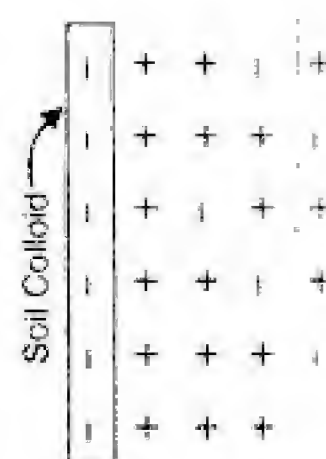


FIG. 5.6. DIFFUSE DOUBLE LAYER

**Colloid repulsion and attraction.** The electric potential  $\psi$  responsible for the formation of the diffuse double layer, reduces with distance [Fig. 5.7 (a)], till, at some distance away from the colloid, free water exists. When two colloids in a suspension approach each other, they will reach an interparticle distance where their double layers interact. The most important result of interaction between double layers is the decrease effected in the charge of each double layer. A decrease in charge results in an increase of the free energy of the double layer. Since systems tend to exist in a state of minimum free energy, the colloids *repulse* each other when their double layer interact (Lambe, 1953). However, there exist long-range attractive forces between colloids in suspension. The attractive forces acting between colloids which are far apart are 'Van der Waal forces' or 'Van der Waal-London Forces'. Debye showed that two dipolar molecules would mutually influence their spatial orientations in such a way that on the average, attraction will occur. The repulsive and attractive energies between particles is a function of the distance between them. Let us now consider the total repulsive plus attractive energy between two particles. If the total potential energy between two particles decreases as they approach each other, there is attraction

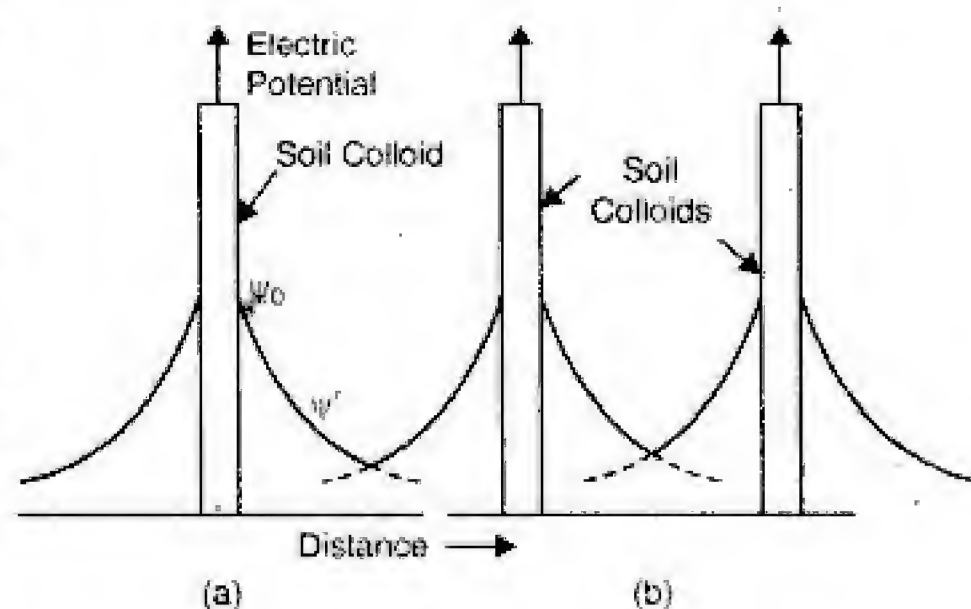


FIG. 5.7. COLLOID POTENTIAL.

between the particles since the force between the particles is the negative derivative of the potential. The two particles will, therefore, approach each other until they touch or stick, or *flocculate*. If, however, the nearing particles increase the potential energy, repulsion or dispersion takes place. As thermal energy brings the particles towards each other, they will repulse and stay in suspension. The various factors in a soil-water system affecting flocculation or dispersion are (Lambe, 1958) : electrolyte concentration, ion valence, dielectric constant, temperature, size of hydrated

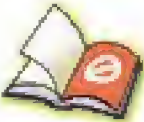


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A clay having flocculent structure has a high voids ratio. When pressure is applied to the deposit, high concentration of stress exists at the points of contact causing the particles to be bent and to slip along the contact surfaces to positions of greater stability and producing a denser arrangement of the particles with a resulting decrease in volume. Remoulding and compacting clays tend to produce a dispersed structure. Similarly, consolidation tends to orient particles into the dispersed arrangement. Pressure applied to a dispersed structure forces some of the double layer water out from between the flakes until they are separated by water with high enough viscosity to support the applied load. Montmorillonite crystals under some conditions may be only  $10\text{\AA}$  ( $\text{\AA} = 10^{-7}\text{ mm}$ ) thick with  $10\text{\AA}$  of adsorbed water next to the surface and  $200\text{\AA}$  of double layer water. Montmorillonite having a dispersed structure under this condition is 97 per cent water and 3 per cent solids by volume.

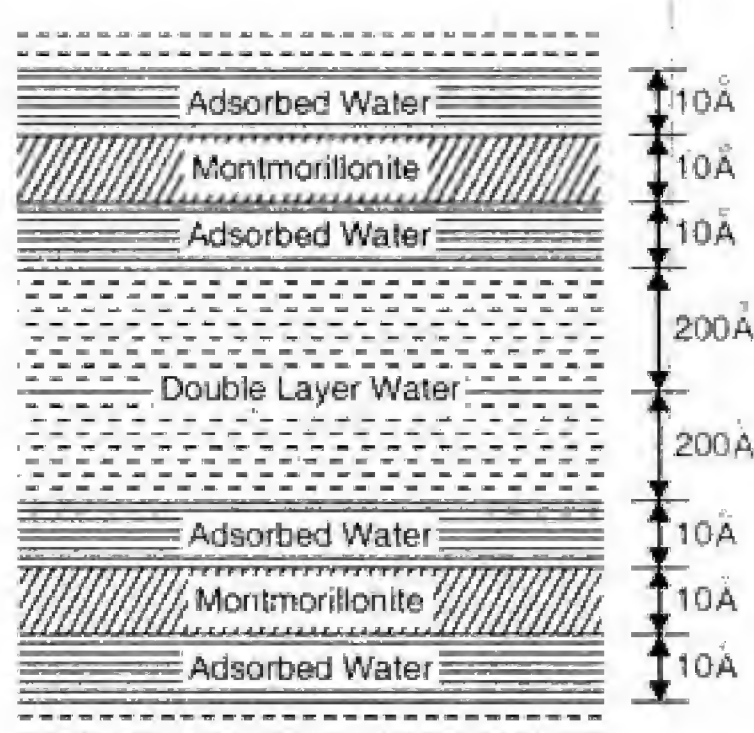


FIG. 5.13. MONTMORILLONITE CRYSTALS WITH ATTACHED WATER (DISPERSED STRUCTURE).

## 5.8. STRUCTURE OF COMPOSITE SOILS

Depending upon the relative proportion of coarse grained and fine grained particles, two types of structures of composite soils can be possible : the *coarse grained skeleton* structure and *cohesive matrix*. In the coarse-grained skeleton, the voids space of the single grained structure is filled with clay particles. The bulky particles form a continuous relatively incompressible frame-work. In the cohesive matrix, the clay content is more so that the bulky particles are not in a position to have particle-to-particle contact. The clay constitutes the load bearing member, and hence the soil formation is relatively more compressible.

## 5.9. CLAY MINERALS

There are two fundamental *building blocks* for the clay mineral structures. One is a *silica tetrahedral unit* [Fig. 5.14 (a)] in which four oxygen or hydroxyls having the configuration of a tetrahedron enclose a silicon atom. The tetrahedra are combined in a sheet structure so the oxygens of the bases of all the tetrahedra are in a common plane,

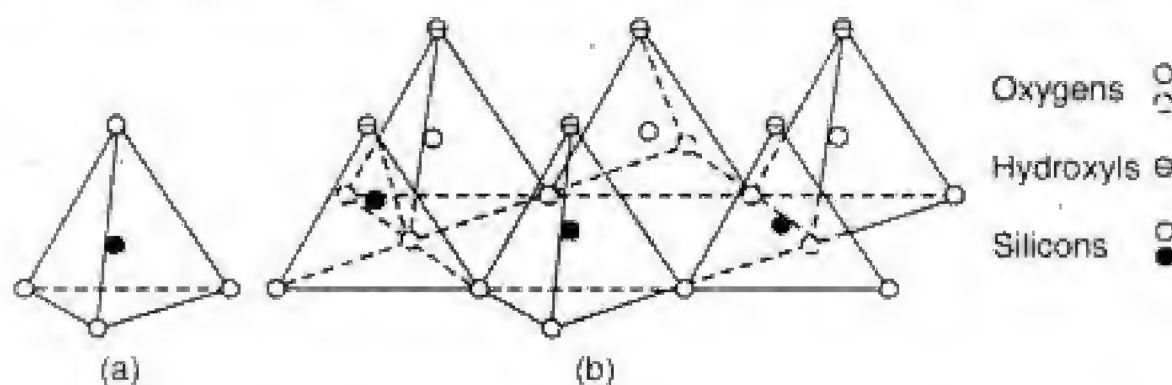
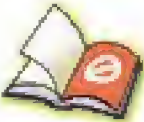


FIG. 5.14. BASIC STRUCTURAL UNITS IN THE SILICASHEET (GRIM, 1959).





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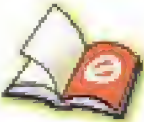


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water has a very definite effect on the cohesion and plasticity characteristics for fine grained soils.

The water in the soil system that is not under significant forces of attraction from the soil particle is pore water. Pore water follows hydrodynamic laws and in general behaves as normal liquid water. Essentially all of the water in sandy soil is pore water; less than half the water in a saturated cohesive soil may be pore water or free water.

### 6.3. CAPILLARY WATER

Capillary water is the soil moisture located within the interstices and voids of capillary size of the soil. Capillary water is held in the interstices of soil due to capillary forces. *Capillary action* or *capillarity* is the phenomenon of movement of water in the interstices of a soil due to capillary forces. The minute pores of soil serve as capillary tubes through which the moisture rises above the ground water table. The capillary forces depend upon various factors such as surface tension of water, pressure in water in relation to atmospheric pressure, and the size and conformation of soil pores.

**SURFACE TENSION.** Surface tension of water is the property which exists in the surface film of water tending to contract the contained volume into a form having a minimum superficial area possible. The molecules on surface of a liquid are attracted by other molecules on the surface and inside the body of the liquid. Because there is no pull from outside, the surface molecules are pulled towards the inside of the liquid mass tending to reduce the surface to a minimum. Thus, in the case of molecules in a drop of water the drops tend to assume a spherical shape. The surface tension  $T_s$  (sometimes designated as the *coefficient of surface tension*) is approximately equal to 72.8 dynes per cm or  $0.728 \times 10^{-6}$  kN/cm at  $20^\circ$  C. The coefficient depends upon the chemical nature of liquids. The surface tension for other common liquids (such as acetone, benzene, petrol etc.) at room temperature ( $20^\circ$  C) are approximately of the order of  $0.29 \times 10^{-6}$  kN/cm. Thus, the surface tension for water is more than double the surface tension for other common liquids. However, the surface tension for mercury is as high  $2.45 \times 10^{-6}$  kN/cm. The formation of curved meniscus around other material inserted in water is due to the surface tension. When a solid or hollow tube, wet with water, is partly inserted vertically in water, the molecules, due to attraction between the molecules of water and the material, climb the solid surface forming a curved meniscus adjacent to the walls of the tube or rod. The tensile strength, due to the attraction of the surface molecules for each other and for the tube wall, supports the water in the meniscus above the horizontal surface (Fig. 6.3).

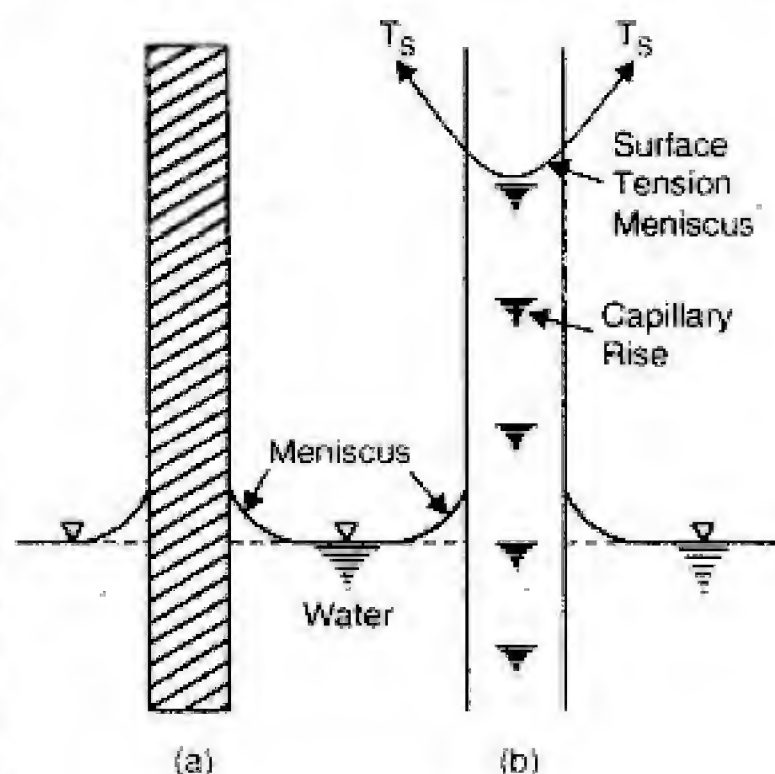


FIG. 6.3. SURFACE TENSION AND FORMATION OF MENISCUS.

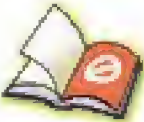


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reduction or negative pressure in the pore water (or the pressure below atmospheric) by which water is retained in a soil mass. It decreases linearly from a maximum value of  $h_c \gamma_w$  at the level of the meniscus to zero value at the free water surface. The pressure deficiency in the held water is also termed as *soil suction* or *suction pressure*. Soil suction is measured by the height  $h_c$  in centimetres to which a water column could be drawn by suction in a soil mass free from external stress. The common logarithm of this height (cm) or pressure ( $\text{g}/\text{cm}^2$ ) is known as the pF value (Schofield, 1935):

$$\text{pF} = \log_{10} (h_c) \quad \dots (6.5)$$

Thus, a pF value of 2 represents a soil suction of 100 cm of water or suction pressure and capillarity of  $100 \text{ g}/\text{cm}^2$ .

### Factors affecting soil suction

Following are some of the factors affecting soil suction :-

1. **Particle size of soil** : Smaller the size of the particles, smaller will be the pore size with small radii of menisci, resulting in greater capillary rise and hence greater suction. This is evident from Table 6.1.

2. **Water content** : Smaller the water content, greater will be the soil suction. Soil suction will attain its maximum value when the soil is dry (see Fig. 6.8 and 6.9)

3. **Plasticity index of soil**. For a given water content, soil suction will be greater in a soil which has greater plasticity index than in the one which has lower plasticity index. Empirical relationship between the suction of remoulded soil and water content for a range of plasticity indices has been established at the Road Research Laboratory England, as shown as Fig. 6.8.

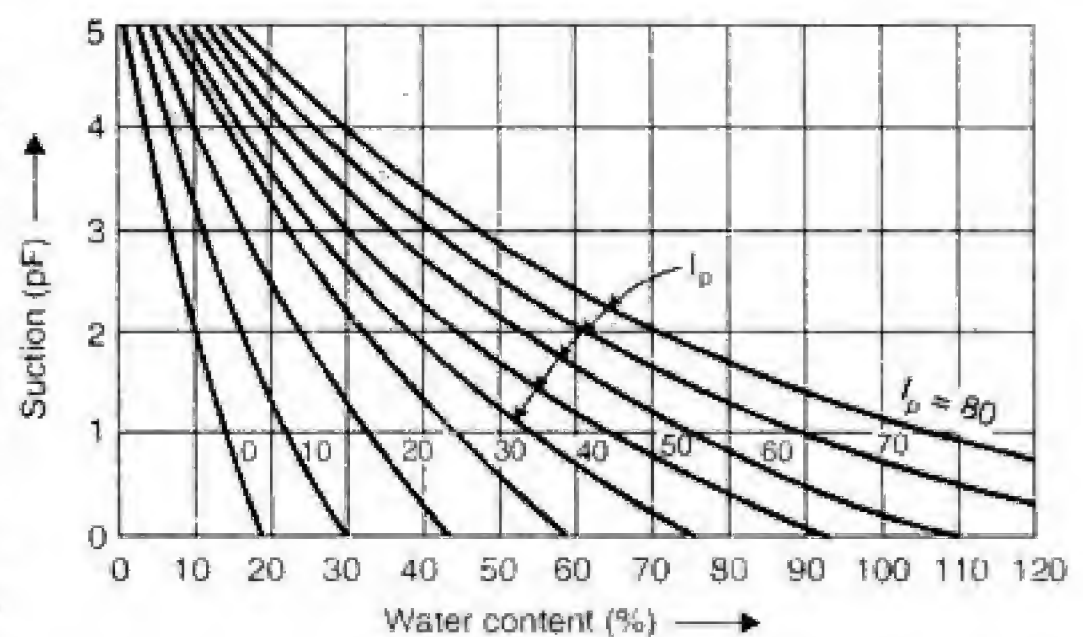


FIG. 6.8. RELATION BETWEEN SOIL SUCTION AND WATER CONTENT AT VARIOUS  $I_p$

4. **History of drying and wetting** : For the same soil, suction is greater during drying cycle than during wetting cycle (see Fig. 6.9).

5. **Soil structure** : The size of interstices in a soil depends upon the structure of the soil. Change in the structure of a soil result in the change in the size of the interstices and hence change in soil suction.

6. **Temperature** : Rise in temperature results in decrease of surface tension ( $T_s$ ) and hence decrease in soil suction. Similarly, fall in temperature results in increase of soil suction.

7. **Denseness of soil** : Increase in denseness of soil results in decrease in the size of the pores of the soil and hence increase in soil suction. At low density, the soil will be relatively loose, with larger size pores, resulting in decrease in soil suction.





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formation of meniscus in the form of plane  $aa$ . Tension in water is zero at this stage. When the water evaporates, the meniscus retreats to positions  $b-b$ ,  $c-c$  etc. causing tension in water and consequent compression in solid grains. The magnitude of tension so developed depends upon the radius of meniscus at that stage. Finally, when the meniscus ( $f-f$ , say) attains its minimum value, compressive forces are maximum and shrinkage is also maximum. Soil at this stage has minimum volume and the water content corresponds to the shrinkage limit for the soil. Further recession of meniscus does not increase the compressive forces, as pores of smaller radius are not available at the that stage.

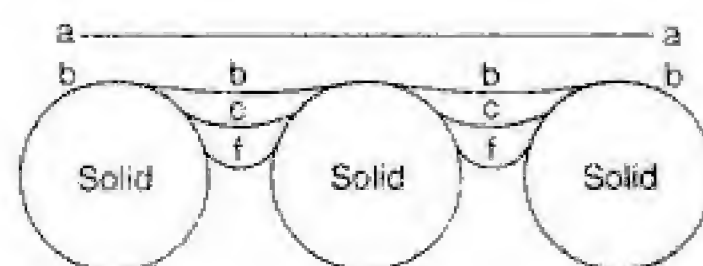


FIG. 6.12. RETREATING OF MENISCUS

The *degree of shrinkage* depends upon several factors such as initial water content, type and amount of clay content, mode and environment, of geological deposition. Shrinkage occurs horizontally as well as vertically, causing vertical shrinkage cracks. Shrinkage is more prominent in clay soils. However, presence of sand and silt-size particles in a clay deposit reduce the total shrinkage.

**Swelling of soil :** When water is added to a soil which has shrunk, menisci are destroyed resulting in tension in the pore water and consequent reduction in compressive stresses in solid grains. This results in elastic expansion of the soil mass, causing *swelling*. However, in clay soils, swelling is caused mainly due to repulsive forces which separate the clay particles, causing volume change. The mechanism of swelling is much more complex than that of shrinkage. Swelling depends upon several factors such as (i) type and amount of clay mineral present in the soil (ii) affinity of clay minerals for water (iii) cation exchange capacity and electrical repulsive forces, (iv) expansion of entrapped air (v) specific surface (vi) structure of soil (vii) elastic rebound of soil grains etc.

As per IS : 2720 - Part 40, 1977, *free swell* of a soil is defined as increase in the volume of a soil without any constraints, on submergence in water. Generally free swell ceases when the water content reaches the plastic limit. When the montmorillonite clay mineral content is very high in a soil, it shows an almost *reversible* swelling and shrinkage on rewetting and redrying. However, when illite and kaolinite clay minerals are present in a clay soil, the soil exhibit large initial volume decrease on drying with only a limited swelling on rewetting.

## 6.7. SLAKING OF CLAY

Consider a mass of clayey soil which has been dried well *below the shrinkage limit*, thus, attaining the minimum volume. When this mass of soil is suddenly immersed in water, it will cause its *slaking*, resulting in its disintegration into a soft wet mass. Such *slaking* is due to the entry of air into the void space during the

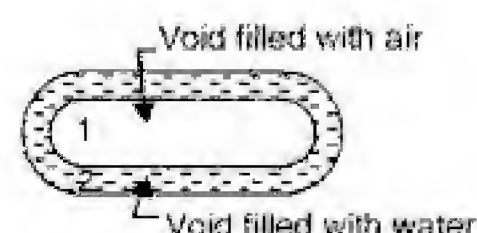


FIG. 6.13. SLAKING OF CLAY

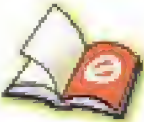


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**Example 6.4.** The Capillary rise in soil A with  $D_{10} = 0.06$  mm is 60 cm. Estimate the capillary rise in soil B with  $D_{10} = 0.1$  mm, assuming the same voids ratio in both the soils.

**Solution :** Let the size of voids be  $d$ .

$$\text{Now } V_s \propto D_{10}^3 \quad \text{and} \quad V_v = e V_s$$

$$\text{But } V_v \propto d^3 \quad \therefore e = \left( \frac{d}{D_{10}} \right)^3 \quad \dots(i)$$

$$\text{For soil A, } d = \frac{0.3084}{h_c} = \frac{0.3084}{60} = 5.14 \times 10^{-3} \text{ cm} = 5.14 \times 10^{-2} \text{ mm}$$

Substituting it in (i), we get

$$e = \left( \frac{d}{D_{10}} \right)^3 = \left( \frac{5.14 \times 10^{-2}}{0.06} \right)^3 = 0.629$$

$$\text{Now for soil B } d = (e)^{1/3} D_{10} = (0.629)^{1/3} \times 0.1 = 0.857 \times 0.1 = 0.0857 \text{ mm} = 0.00857 \text{ cm.}$$

Alternatively, since voids ratio for both soils is equal, we get from (i)

$$\left( \frac{d}{D_{10}} \right)_A = \left( \frac{d}{D_{10}} \right)_B$$

$$\therefore (d)_B = \left( \frac{d}{D_{10}} \right)_A \times (D_{10})_B = \frac{5.14 \times 10^{-2}}{0.06} \times 0.1 = 0.0857 \text{ mm} = 0.00857 \text{ cm}$$

$$\text{Hence } (h_c)_B = \frac{0.3084}{0.00857} = 36 \text{ cm}$$

**Example 6.5.** A glass vessel is filled with water, as shown in Fig. 6.17. A fully developed meniscus has formed in a small hole of diameter  $d_1 = 0.02$  cm in the wall of the vessel. Another hole of diameter  $d_2$  exists in the lower wall. What is the greatest value which  $d_2$  may have? If  $d_2 = d_1$ , find the contact angle in the lower hole.

**Solution.**

(a) As is evident from Fig. 6.17, the capillary heights supported by both the holes will be in opposite directions. However, the algebraic sum of the heights of water supported by them is equal to 9 cm.

$$\text{i.e. } h_{c1} - h_{c2} = 9 \quad \text{or} \quad h_{c2} = h_{c1} - 9 \quad \dots(1)$$

$$\text{Now } h_{c1} = \frac{0.3084}{0.02} = 15.42 \text{ cm (when } \alpha \approx 0 \text{ )}$$

Thus, the upper hole can support a height of 15.42 cm

$$h_{c2} = 15.42 - 9 = 6.42 \text{ cm}$$

For full meniscus to be developed in the lower hole.

$$d_2 = \frac{0.3084}{6.42} = 0.048 \text{ cm}$$

This is the maximum value of  $d_2$ . If  $d_2$  is increased,  $h_{c2}$  will be decreased, the corresponding value of  $h_{c1}$  ( $= 9 + h_{c2}$ )

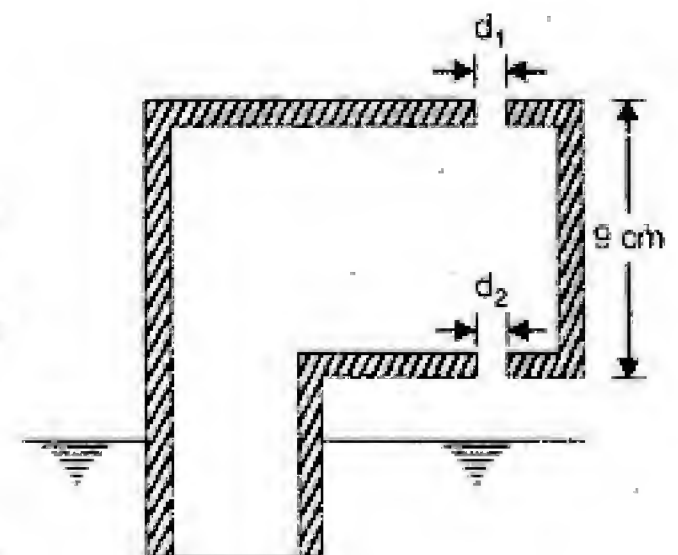


FIG. 6.17



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the ground surface. What will be the increase in the effective pressure if the soil gets saturated by capillarity upto a height of 1 m above the water table ? Assume  $G = 2.65$ .

**Solution :** Height of sand layer above water table  $= Z_1 = 4$  m

Height of saturated layer  $= 12 - 4 = 8$  m

Depth of point X, where pressure is to be computed  $= 10$  m

Height of saturated layer above X  $= Z_2 = 10 - 4 = 6$  m

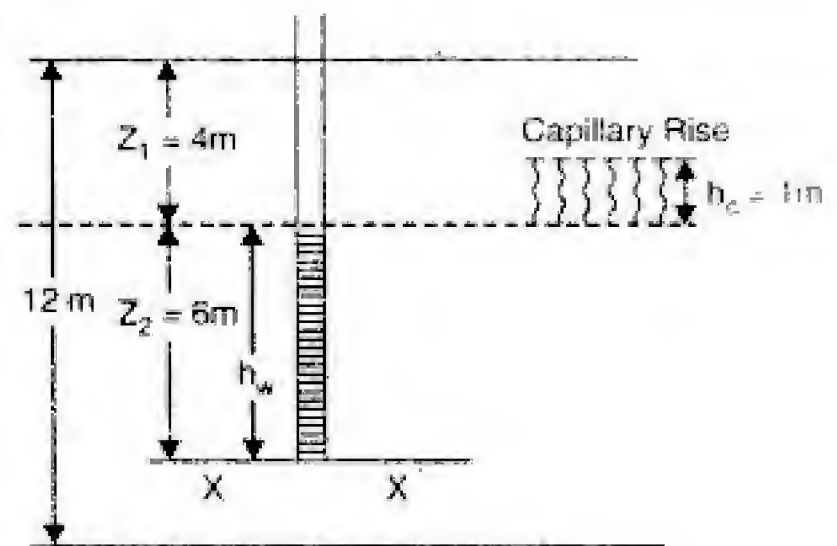


FIG. 6.22

Now 
$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+0.7} = 15.29 \text{ kN/m}^3$$

(i) For sand above water table,

$$e = \frac{wG}{S_r} \quad \text{Hence } w = \frac{e S_r}{G} = \frac{0.7 \times 0.5}{2.65} = 0.132$$

$$\gamma_1 = \gamma_d (1 + w) = 15.29 \times 1.132 = 17.31 \text{ kN/m}^3.$$

(ii) For saturated sand below water table

$$w_{sat} = \frac{e}{G} = \frac{0.7}{2.65} = 2.64$$

$$\gamma_2 = \gamma_d (1 + w_{sat}) = 15.29 \times 1.264 = 19.33 \text{ kN/m}^3$$

$$\gamma_2' = 19.33 - 9.81 = 9.52 \text{ kN/m}^3.$$

*Effective pressure at X*

$$\sigma = Z_1 \gamma_1 + Z_2 \gamma_2 = 4 \times 17.31 + 6 \times 19.33 = 185.22 \text{ kN/m}^2$$

$$u = h_w \gamma_w = 6 \times 9.81 = 58.86 \text{ kN/m}^2$$

$$\therefore \sigma' = \sigma - u = 185.22 - 58.86 = 126.36 \text{ kN/m}^2$$

Alternatively,  $\sigma' = Z_1 \gamma_1 + Z_2 \gamma_2' = (4 \times 17.31) + (6 \times 9.52) = 126.36 \text{ kN/m}^2$

*Effective stress at X after capillary rise*

$$\begin{aligned} \sigma' &= 3 \gamma_1 + (6 + 1) \gamma_2' + h_c \gamma_w = (3 \times 17.31) + (7 \times 9.52) + (1 \times 9.81) \\ &= 128.38 \text{ kN/m}^2 \end{aligned}$$

$$\text{Increase in pressure} = 128.38 - 126.36 = 2.02 \text{ kN/m}^2$$

**Example 6.7.** A 10 m thick bed of sand is underlain by a layer of clay of 6 m thickness. The water table which was originally at the ground surface is lowered by drainage to a depth of 4 m, whereupon the degree of saturation above the lowered water table reduces to 20%. Determine the increase in the magnitude of the vertical effective pressure at the middle of the clay layer due to lowering of water table. The saturated unit weights of sand and clay are respectively  $20.6 \text{ kN/m}^3$  and  $17.6 \text{ kN/m}^3$ , and the dry unit weight of sand is  $16.7 \text{ kN/m}^3$ .





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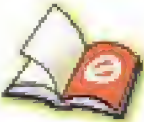


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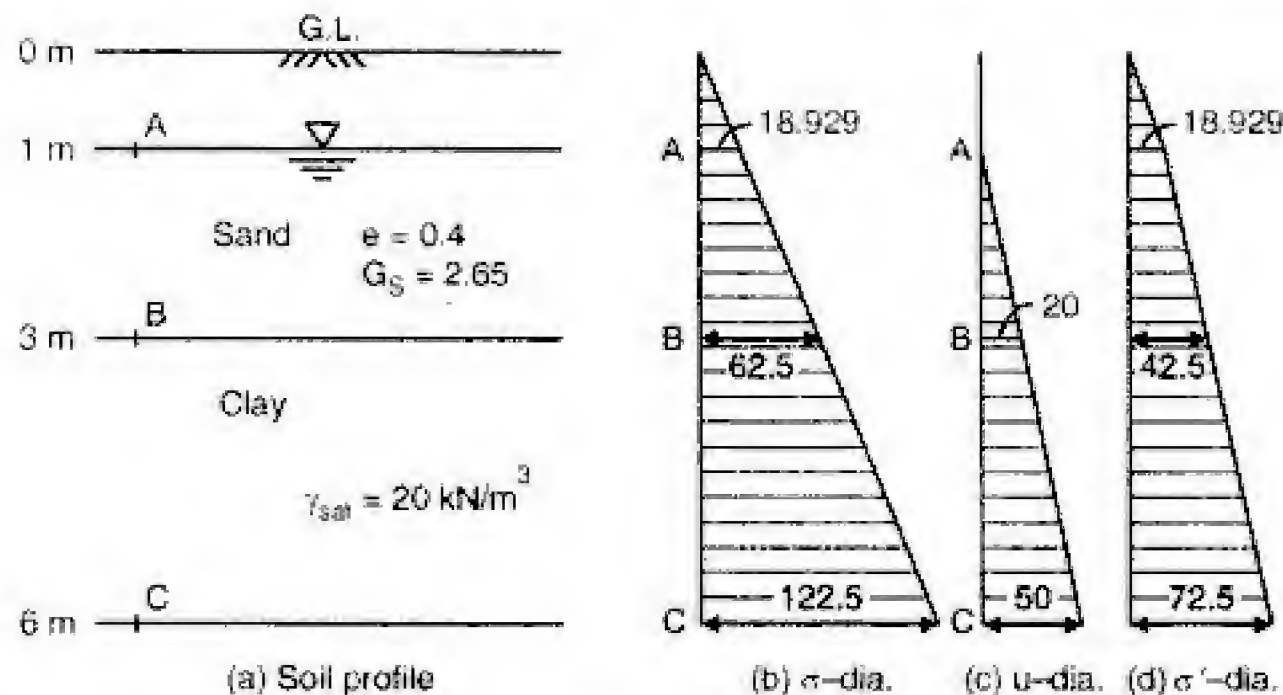


FIG. 6.31

$$\gamma' = 21.786 - 10 = 11.786 \text{ kN/m}^3$$

For clay :  $\gamma_{\text{sat}} = 20 \text{ kN/m}^3$  (Given)

At A :  $\sigma = 18.929 \times 1 = 18.929 \text{ kN/m}^2$ ;  $u = 0$ ;  $\sigma' = 18.929 - 0 = 18.929 \text{ kN/m}^2$

At B :  $\sigma = 18.929 + 21.786 \times 2 = 62.5 \text{ kN/m}^2$   
 $u = 10 \times 2 = 20 \text{ kN/m}^2$ ;  $\sigma' = 62.5 - 20 = 42.5 \text{ kN/m}^2$

At C :  $\sigma = 62.5 + 20 \times 3 = 122.5 \text{ kN/m}^2$   
 $u = 10 \times 5 = 50 \text{ kN/m}^2$ ;  $\sigma' = 122.5 - 50 = 72.5 \text{ kN/m}^2$

**Example 6.16.** A layer of saturated clay 5 m thick is overlain by sand 4.0 m deep. The water-table is 3 m below the top surface. The saturated weights of clay and sand are  $18 \text{ kN/m}^3$  and  $20 \text{ kN/m}^3$ , respectively. Above the water table, the unit weight of sand is  $17 \text{ kN/m}^3$ . Calculate the effective pressure on a horizontal plane at a depth of 9 m below the ground surface. What will be the increase in the effective pressure at 9 m, if the soil gets saturated by capillary, upto height of 1 m above the water table?

$\gamma_w = 9.81 \text{ kN/m}^3$ . (Gate Exam. 1999)

**Solution :** Refer Example 6.6.

$$\begin{aligned} \sigma' \text{ at } A &= 3 \gamma_{\text{sand}} + 1 \gamma_{\text{sat., sand}} + 5 \gamma'_{\text{clay}} = 3 \times 17 + 1 (20 - 9.81) + 5 (18 - 9.81) \\ &= 51 + 10.19 + 40.95 = 102.14 \text{ kN/m}^2 \end{aligned}$$

If the soil gets saturated by capillarity, then

$$\begin{aligned} \sigma' \text{ at } A &= 2 \gamma_{\text{sand}} + 1 \gamma_{\text{sat., sand}} + \gamma'_{\text{sand}} + 5 \gamma'_{\text{clay}} = 2 \times 17 + 1 (20) + 1 (20 - 9.81) + 5 (18 - 9.81) \\ &= 34 + 20 + 10.19 + 40.95 = 105.14 \text{ kN/m}^2 \end{aligned}$$

**Alternatively :**  $\sigma'_A = 2 \times 17 + (1 + 1) (20 - 9.81) + 1 \times 9.81 + 5 (18 - 9.81)$

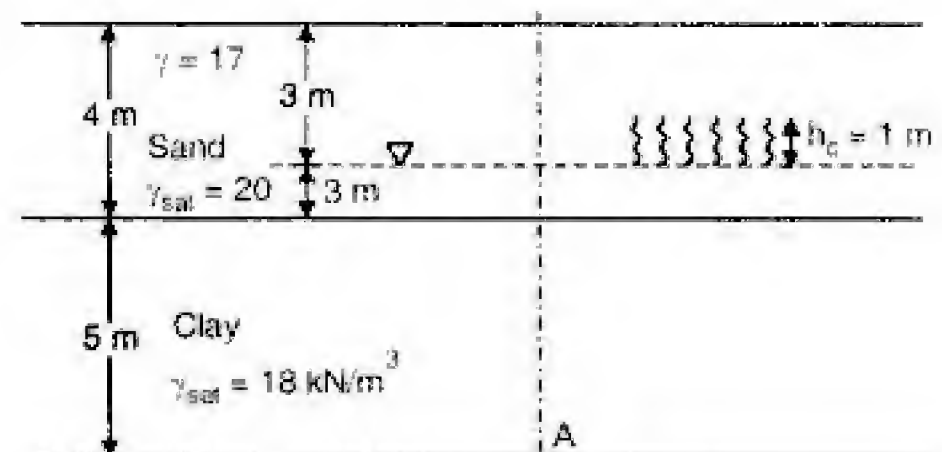


FIG. 6.32





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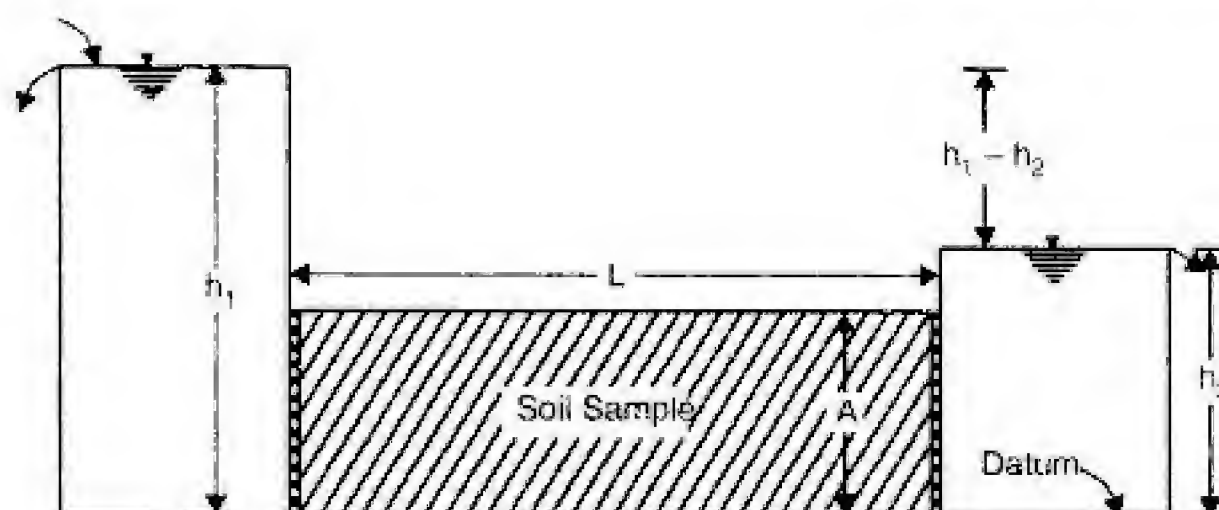


FIG. 7.1. FLOW OF WATER THROUGH SOIL.

or 
$$v = \frac{q}{A} = k i \quad \dots(7.2)$$

where  $q$  = discharge per unit time  
 $A$  = total cross-sectional area of soil mass, perpendicular to the direction of flow  
 $i$  = hydraulic gradient  
 $k$  = Darcy's coefficient of permeability  
 $v$  = velocity of flow, or average *discharge velocity*.

If a soil sample of length  $L$  and cross-sectional area  $A$ , is subjected to differential head of water,  $h_1 - h_2$ , the hydraulic gradient  $i$  will be equal to  $\frac{h_1 - h_2}{L}$  and, we have

$$q = k \frac{h_1 - h_2}{L} A \quad \dots(7.1 a)$$

From Eq. 7.2, when hydraulic gradient is unity,  $k$  is equal to  $v$ . Thus, the *coefficient of permeability*, or simply *permeability*, is defined as the average velocity of flow that will occur through the total cross-sectional area of soil under unit hydraulic gradient. The dimensions of the coefficient of permeability  $k$  are the same as those of velocity. It is usually expressed as cm/sec or m/day or feet/day.

Table 7.1 gives some typical values of co-efficient of permeability of various soils.

17.1 TYPICAL VALUES OF  $k$ 

Soil Type	Co-efficient of permeability cm/sec
Clean gravel	1.0 and greater
Clean sand (coarse)	$1.0 - 1 \times 10^{-2}$
Sand (mixture)	$1 \times 10^{-2} - 5 \times 10^{-2}$
Fine sand	$5 \times 10^{-2} - 1 \times 10^{-3}$
Silty sand	$2 \times 10^{-3} - 1 \times 10^{-4}$
Silt	$5 \times 10^{-4} - 1 \times 10^{-5}$
Clay	$1 \times 10^{-6}$ and smaller





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**Effect of shape of the capillary tube.** Eqs. 7.10 and 7.11 are valid for *circular* capillary tube only. In engineering hydraulics, the velocity is generally designated in terms of the *hydraulic radius*  $R_H$ , which is defined as the ratio of area to the wetted perimeter.

$$\text{For circular tube} \quad R_H = \frac{\pi R^2}{2 \pi R} = \frac{R}{2} \quad \text{or} \quad R = 2 R_H$$

Substituting this in Eq. 7.10, we get

$$q_{cir} = \frac{1}{2} \frac{\gamma_w R_H^2}{\eta} i a \quad \dots(7.12)$$

Similarly, the rate of flow between two flat, closely spaced, parallel plates, can be found to be given by

$$q_{pl} = \frac{1}{3} \frac{\gamma_w R_H^2}{\eta} i a \quad \dots(7.13)$$

Hence, we can conclude that for the capillary tube of any other given geometrical shape of cross-section, the flow may be expressed as

$$q = C_s \frac{\gamma_w R_H^2}{\eta} i a \quad \dots(7.14)$$

where  $C_s$  = shape constant.

Fig. 7.3 shows the shape of a irregular capillary voids. Let the total cross-sectional area of the soil mass be  $A$ , and  $n$  be the porosity.

$$\therefore a = \text{area of flow passage} = nA$$

$$\therefore q = \left( C_s \frac{\gamma_w R_H^2}{\eta} n \right) i A = \left( C_s \frac{\gamma_w R_H^2}{\eta} \frac{e}{1+e} \right) i A \quad \dots(7.15)$$

**Hydraulic mean radius in soil pores.** Kozeny's (1927) concept of hydraulic mean radius in soils leads to the following expression :

$$R_H = \frac{A}{P} = \frac{AL}{PL}$$

If  $V_s$  is the volume of solids in a soil mass having voids ratio  $e$ , the volume of the flow channel ( $= AL$ ) will be  $eV_s$ . The total surface area of flow channel ( $= PL$ ) is equal to the total *surface area*  $A_s$  of the soil grains.

$$\therefore R_H = \frac{AL}{PL} = \frac{e V_s}{A_s} \quad \dots(7.16)$$

Let  $D_s$  = diameter of the spherical grain which has the same ratio of volume to surface area as holds collectively for the grains in a given soil.

$$\therefore \frac{V_s}{A_s} = \frac{\frac{1}{6} \pi D_s^3}{\pi D_s^2} = \frac{D_s}{6} \quad ; \quad \therefore R_H = \frac{D_s}{6} e \quad \dots(7.17 \ a)$$

Substituting this in Eq. 7.15,

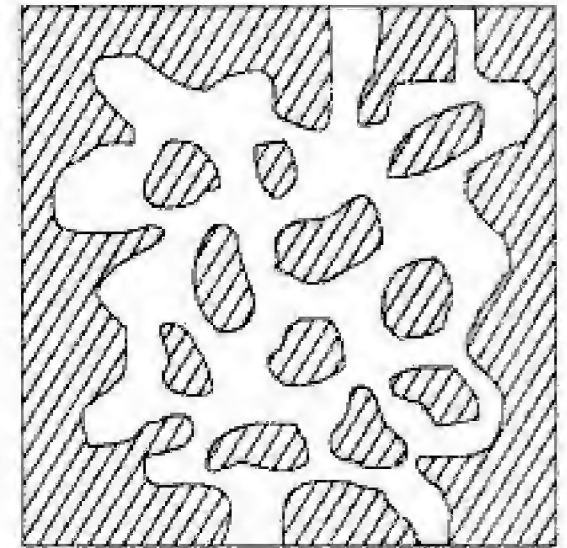


FIG. 7.3. TUBE OF IRREGULAR SHAPE..





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$$[K] = \left[ \frac{L}{T} \right] \left[ \frac{FT}{L^2} \right] \left[ \frac{L^3}{F} \right] = [L^2]$$

Hence  $K$  has the dimensions of area. The units of  $K$  are :  $\text{mm}^2$ ,  $\text{cm}^2$ ,  $\text{m}^2$  or darcy.

$$1 \text{ darcy} = 0.987 \times 10^{-8} \text{ cm}^2.$$

It is interesting to note that for a given voids ratio and structural arrangement of particles, the coefficient of absolute permeability ( $K$ ) is constant, irrespective of type/properties of fluid.

## 7.8 DETERMINATION OF COEFFICIENT OF PERMEABILITY

The coefficient of permeability can be determined by the following methods :

- (a) *Laboratory methods*
  - (1) Constant head permeability test.
  - (2) Falling head permeability test.
- (b) *Field methods*
  - (1) Pumping-out tests. (2) Pumping-in tests.
- (c) *Indirect methods*
  - (1) Computation from grain size or specific surface.
  - (2) Horizontal capillarity test
  - (3) Consolidation test data.

Permeability can be determined in the laboratory by direct measurement with the help of *permeameters*, by allowing the water to flow through soil sample either under *constant head* or under *variable head*. Permeability can also be determined directly by field test, described in chapter 8 (well hydraulics). The indirect method of computing the permeability from consolidation test data has been explained in chapter 15.

### Empirical formulae for determination of coefficient of permeability ( $k$ )

Permeability can also be computed from several empirical formulae given below.

#### 1. Jaky's formula

Jaky (1944) found that a fair estimate of the order of magnitude of  $k$  can be obtained for all soils from the formula

$$k = 100 D_m^2 \quad \dots(7.25 \text{ a})$$

where  $D_m$  denotes grain size (in cm) that occurs with the greatest frequency.

#### 2. Allen Hazen's formula

$$k = C D_{10}^4 \quad \dots(7.25 \text{ b})$$

where  $C$  is a constant, which is taken approximately equal to 100 when  $D_{10}$  is expressed in cm.

#### 3. Terzaghi's formula

Terzaghi (1955) developed the following formula for fairly uniform sands, which reflects the effect of grain size and void ratio :





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compaction, the 3 cm plug and 3 cm collar are removed and, after putting the fine mesh gauge etc., perforated base plate is fixed over it. The mould is turned upside down, the plug and the collar are removed, and the top perforated plate and top cap are fixed.

To compact the specimen dynamically, using the rod temper, the mould is fixed upside down on the dynamic compaction plate, and the collar is fixed to its other end. The wet soil of pre-calculated quantity is then compacted into the mould by means of the rod temper, in two or three layers. After compaction, the collar is removed and, after placing the fine mesh gauge, the perforated base plate is fixed. The mould assembly is then turned upside down, the compaction base plate is detached, and the top cap is fixed.

Alternatively, if permeability at Proctor's maximum dry density and at a moulding water content equal to the optimum value is required, first the maximum dry density and optimum water content is determined. (This can be done by Jodhpur Mini compactor test, Proctor test or by compaction in permeameter mould itself). The soil is then compacted at the optimum water content in two layers in the 0.3 litre permeameter mould (mould assembly as described in the above para) with 15 blows of 2.5 kg DRT given to each layer. After the compaction, the compaction collar is removed, the excess soil is trimmed off, and the perforated base plate is fixed, as described in the above para.

**Example 7.1.** Calculate the co-efficient of permeability of a soil sample, 6 cm in height and 50 cm<sup>2</sup> in cross-sectional area, if a quantity of water equal to 430 ml passed down in 10 minutes, under an effective constant head of 40 cm.

On oven-drying, the test specimen has mass of 498 g. Taking the specific gravity of soil solids as 2.65, calculate the seepage velocity of water during the test.

**Solution :** Given :  $Q = 430$  ml ;  $t = 10 \times 60 = 600$  seconds

$$A = 50 \text{ cm}^2 ; L = 6 \text{ cm} ; h = 40 \text{ cm}$$

$$\begin{aligned} \text{From Eq. 7.26, } k &= \frac{Q}{t} \cdot \frac{L}{h} \cdot \frac{1}{A} = \frac{430}{600} \times \frac{6}{40} \times \frac{1}{50} = 2.15 \times 10^{-3} \text{ cm/sec} \\ &= 2.15 \times 10^{-3} \times 864 = 1.86 \text{ m/day (Since 1 cm/sec = 864 m/day)} \end{aligned}$$

$$\text{Now } v = \frac{q}{A} = \frac{430}{600 \times 50} = 1.435 \times 10^{-2} \text{ cm/sec}$$

$$\text{Alternatively, } v = ki = 2.15 \times 10^{-3} \times \frac{40}{6} = 1.435 \times 10^{-2} \text{ cm/sec}$$

$$\text{Now } \rho_d = \frac{M_d}{V} = \frac{498}{50 \times 6} = 1.66 \text{ g/cm}^3 \quad \therefore e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.65 \times 1}{1.66} - 1 = 0.595$$

$$\therefore n = \frac{e}{1 + e} = \frac{0.595}{1.595} = 0.373$$

$$\therefore v_s = \frac{v}{n} = \frac{1.435 \times 10^{-2}}{0.373} = 3.85 \times 10^{-2} \text{ cm/sec}$$

**Example 7.2.** In a falling head permeameter test, the initial head ( $t = 0$ ) is 40 cm. The head drops by 5 cm in 10 minutes. Calculate the time required to run the test for



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**Example 7.4.** A constant head permeability test was run on a sand sample 16 cm in length and 60 cm<sup>2</sup> in cross-sectional area. Porosity was  $n_1 = 40\%$ . Under a constant head of 30 cm, the discharge was found to be 45 cm<sup>3</sup> in 18 seconds. Calculate the coefficient of permeability. Also, determine the discharge velocity and seepage velocity during the test. Estimate the permeability of the sand for a porosity of  $n_2 = 35\%$ .

**Solution**

From Eq. 7.26, 
$$k = \frac{Q}{t} \cdot \frac{L}{h} \cdot \frac{1}{A} = \frac{45}{18} \times \frac{16}{30} \times \frac{1}{60} = 2.22 \times 10^{-2} \text{ cm/s}$$

Discharge velocity, 
$$v = k i = k \frac{h}{l} = 2.22 \times 10^{-2} \times \frac{30}{16} = 4.17 \times 10^{-2} \text{ cm/s}$$

Seepage velocity, 
$$v_s = \frac{v}{n} = \frac{4.17 \times 10^{-2}}{0.4} = 10.42 \times 10^{-2} \text{ cm/s}$$

Again, from Eq. 7.23 (a)

$$\frac{k_1}{k_2} = \frac{e_1^3}{1 + e_1} \cdot \frac{1 + e_2}{e_2^3} = \frac{n_1^3}{(1 - n_1)^2} \cdot \frac{n_2^3}{(1 - n_2)^2}$$

or 
$$k_2 = k_1 \frac{\frac{n_2^3}{(1 - n_2)^2}}{\frac{n_1^3}{(1 - n_1)^2}} = 2.22 \times 10^{-2} \times \frac{\frac{0.35^3}{(1 - 0.35)^2}}{\frac{0.4^3}{(1 - 0.4)^2}} = 1.26 \times 10^{-2} \text{ cm/s}$$

**Example 7.5.** Permeability tests were performed on a soil sample, under different voids ratio and different temperatures and the following results were obtained.

Test No.	Voids ratio (e)	Temperature °C	k(cm/s)
1	0.65	25°	$0.4 \times 10^{-4}$
2	1.02	40°	$1.9 \times 10^{-4}$

Estimate the coefficient of permeability at a temperature of 20° C for a voids ratio of 0.80.

Given the following physical properties of water:

At 20° C,  $\eta = 10.09 \times 10^{-4}$  and  $\rho_w = 0.998 \text{ g/cm}^3$

At 25° C,  $\eta = 8.95 \times 10^{-4} \text{ g sec/cm}^2$  and  $\rho_w = 0.997 \text{ g/cm}^3$

At 40° C,  $\eta = 6.54 \times 10^{-4} \text{ g sec/cm}^2$  and  $\rho_w = 0.992 \text{ g/cm}^3$

**Solution**

**Step 1 :** Let us first convert both the test results to a temperature of 20° C.

From Eq. 7.22, 
$$\frac{k_1}{k_2} = \frac{\eta_2}{\eta_1} \cdot \frac{\rho_{w1}}{\rho_{w2}}$$

For first test, 
$$(k_1)_{20} = 0.4 \times 10^{-4} \times \frac{8.95 \times 10^{-4}}{10.09 \times 10^{-4}} \times \frac{0.998}{0.997} = 0.355 \times 10^{-4} \text{ cm/s}$$



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Let  $Z_1 = 4$ ,  $Z_2 = 1$  and  $Z_3 = 2$  units.  $Z = 4 + 1 + 2 = 7$  units

$$k_x = \frac{(2 \times 4) + (1 \times 1) + (2 \times 4)}{7} = \frac{17}{7} = 2.43 \quad ; \quad k_z = \frac{7}{\frac{4}{3} + \frac{1}{1} + \frac{2}{4}} = \frac{7}{3.5} = 2$$

$$\therefore k_x > k_z.$$

**Example 7.6.** A stratified soil deposit consists of four layers of equal thickness. The co-efficient of permeability of the second, third and fourth layers are respectively  $\frac{1}{3}$  rd,  $\frac{1}{2}$  and twice of the coefficient of permeability of the top layer. Compute the average permeabilities of the deposit, parallel and perpendicular to the direction of the stratification in terms of the permeability of the top layer.

**Sloution :** Let the thickness of the top layer be  $Z$  and its permeability be  $k$ .

$\therefore$  Total thickness of deposit  $= 4Z$

Now  $k_1 = k$  ;  $k_2 = \frac{1}{3}k$  ;  $k_3 = \frac{k}{2}$  ;  $k_4 = 2k$

$$\therefore k_x = \frac{Z \cdot k + \frac{Zk}{3} + \frac{Zk}{2} + Z \cdot 2k}{4Z} = \frac{23}{24}k$$

$$k_z = \frac{4Z}{\frac{Z}{k} + \frac{3}{k}Z + \frac{2}{k}Z + \frac{Z}{2k}} = \frac{8}{13}k$$

**Example 7.7.** Fig. 7.10 shows an aquifer inclined at  $12^\circ$  to the horizontal. Two observation wells, dug upto the aquifer, at a horizontal distance of 80 m show a difference of 6 m in the water levels. Taking coefficient of permeability of aquifer soil as 1.2 mm/sec, determine the discharge through the aquifer, per unit width. The thickness of aquifer normal to the direction of flow is 3.2 m.

**Solution :** Length of travel ( $L$ ) between wells A and B

$$= \frac{80}{\cos 12^\circ} = 81.787 \text{ m}$$

$$\therefore i = \frac{h}{L} = \frac{6}{81.787} = 0.0734$$

Now, from Darcy law, flow per unit width of aquifer is given by

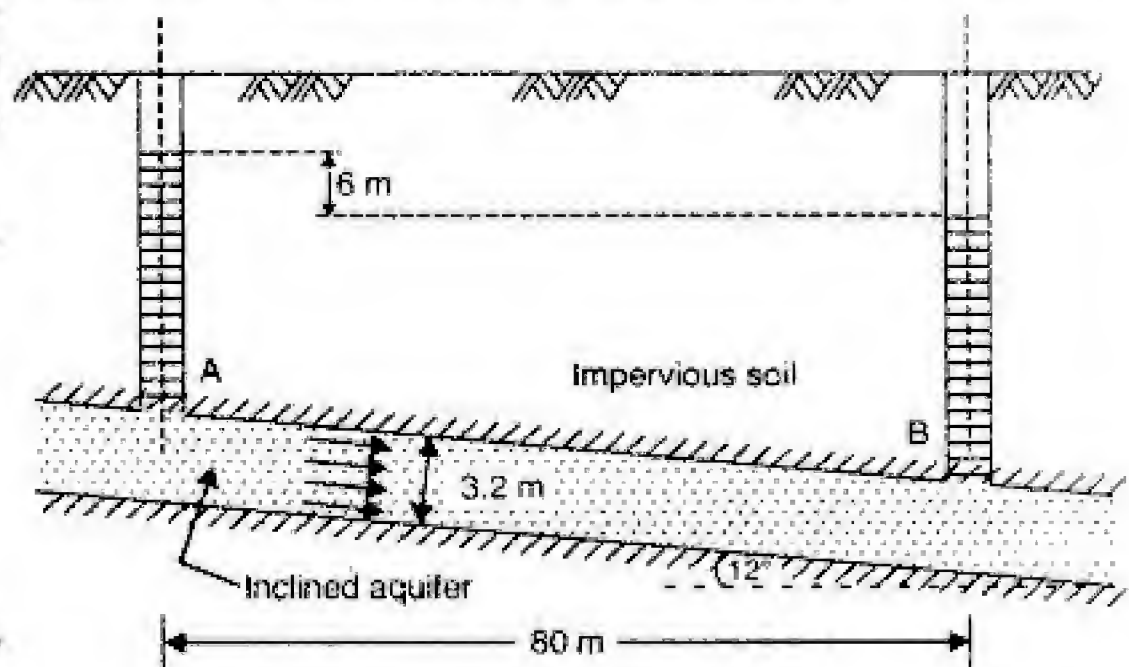


FIG. 7.10

$$q = k i A = 1.2 \times 10^{-3} (0.0734) (3.2 \times 1) = 0.288 \times 10^{-3} \text{ m}^3/\text{sec} \\ = 0.288 \text{ lit/sec}$$



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## 7.15. LABORATORY EXPERIMENTS

### EXPERIMENT 14 : DETERMINATION OF PERMEABILITY BY CONSTANT HEAD TEST

**Object and scope.** The object of the experiment is to determine the coefficient of permeability of soil in the laboratory by constant head test using Jodhpur Permeameter.

**Materials and equipment.** (i) Jodhpur permeameter complete with all accessories, (ii) De-aired water, (iii) Balance to weigh to 1 g, (iv) 4.75 mm and 2 mm IS sieves, (v) Mixing pan or basin, (vi) Stop watch, (vii) Graduated measuring cylinder, (viii) Metre scale, (ix) Beaker, (x) Thermometer, (xi) Containers for water content determination, (xii) Straight edge or trimming knife.

#### Test procedure

##### (a) *Preparation of statically compacted remoulded specimen*

1. Take 800 to 1000 g of representative specimen of soil and mix water to it so that its water content is raised to the optimum water content for the soil determined by Proctor's test. If permeability is to be determined at any other water content, raise the water content of the soil to the desired value. Leave the soil mix for some time in air-tight container.

2. For the given volume ( $V$ ) of the mould, calculate the mass ( $M$ ) of the soil mix so as to give the desired dry density ( $\rho_d$ ), using the following expression :

$$M = \rho_d (1 + w) V$$

Take the mass of the above soil accurate to 1 g.

3. Assemble permeameter for static compaction. For this, attach the 3 cm collar to the bottom end of the 0.3 litre mould and 2.5 cm collar to its top end. Support the mould assembly over the 2.5 cm end plug with the 2.5 cm collar resting on the split collar kept around the 2.5 cm end plug. The 0.3 litre mould should be lightly greased from inside.

4. Put the weighed quantity of soil (step 2) into the mould assembly. Insert the top 3 cm end plug into the top collar. The soil may be tamped with hand while being poured into the mould. Keep the entire assembly into a compression machine and remove the split collar. Apply compressive force on the assembly till the flanges of both the end plugs touch the corresponding collars.

5. Maintain the load for about 1 minute and then release it. Remove the top 3-cm plug and collar. Place a filter paper or fine wire gauge on the top of the specimen and fix the perforated base plate on it.

6. Turn the mould assembly upside down and remove the 2.5 cm end plug and collar. Place the top perforated plate on the top of the soil specimen and fix the top can on to it, after inserting the sealing gasket.

The specimen is now ready for the permeability test.

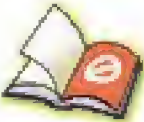
##### (b) *Preparation of dynamically compacted remoulded specimen*

1. Take 800 to 1000 g of representative specimen of soil and raise its water content to the optimum water content. Leave the soil mix in an air-tight container for some time.

2. Assemble the permeameter for dynamic compaction. For this, grease the mould lightly from inside and place it upside down on the dynamic compaction base. Find the mass of the assembly accurate to 1 g. Put the 3 cm collar to the other end.

3. Compact the wet soil mix in two layers, with 15 blows of the 2.5 kg dynamic ramming tool, given to each layer. Remove the collar and trim off the excess soil. Find the mass of mould assembly with soil. The difference of the two masses taken in steps (2) and (3) would give the mass ( $M$ ) of the soil compacted.

4. Place filter paper or fine wire mesh on the top of soil specimen and fix the perforated base plate on to it.



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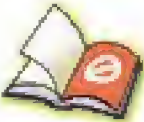


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2. The flow is horizontal and uniform everywhere in the vertical section.
3. Aquifer is homogeneous, isotropic and of infinite aerial extent.
4. The well penetrates and receives water from the entire thickness of the aquifer.
5. The co-efficient of transmissibility is constant at all places and at all times.
6. Natural ground water regime affecting an aquifer remains constant with time.
7. Flow is laminar and Darcy's law is valid.

Out of these, assumptions 1, 2 and 7 are of particular importance. The flow is not horizontal, especially near the well. Also, the piezometric surface attains greater slope as it approaches the well boundary, with the result that assumption 1 is an approximation. Due to these reasons, the parabolic form of piezometric surface computed from the Dupuit's theory deviates from the observed surface. This deviation is large, resulting in the formation of a *seepage face*. In addition to these, the velocity near the well increases, and the flow no longer remains laminar. Thus, Darcy's law equation is not valid near the well face.

2. **Confined aquifer.** Fig. 8.3 shows a well fully penetrating a confined or artesian aquifer. Let  $(x, y)$  be the coordinates of any point  $P$  on the drawdown curve, measured with respect to the origin  $O$ . Then, from Darcy's law, flow crossing a vertical plane through  $P$  is given by  $q = k i_x A_x$ .

where  $A_x$  = cross-sectional area of flow, measured at  $P = 2\pi \times b$

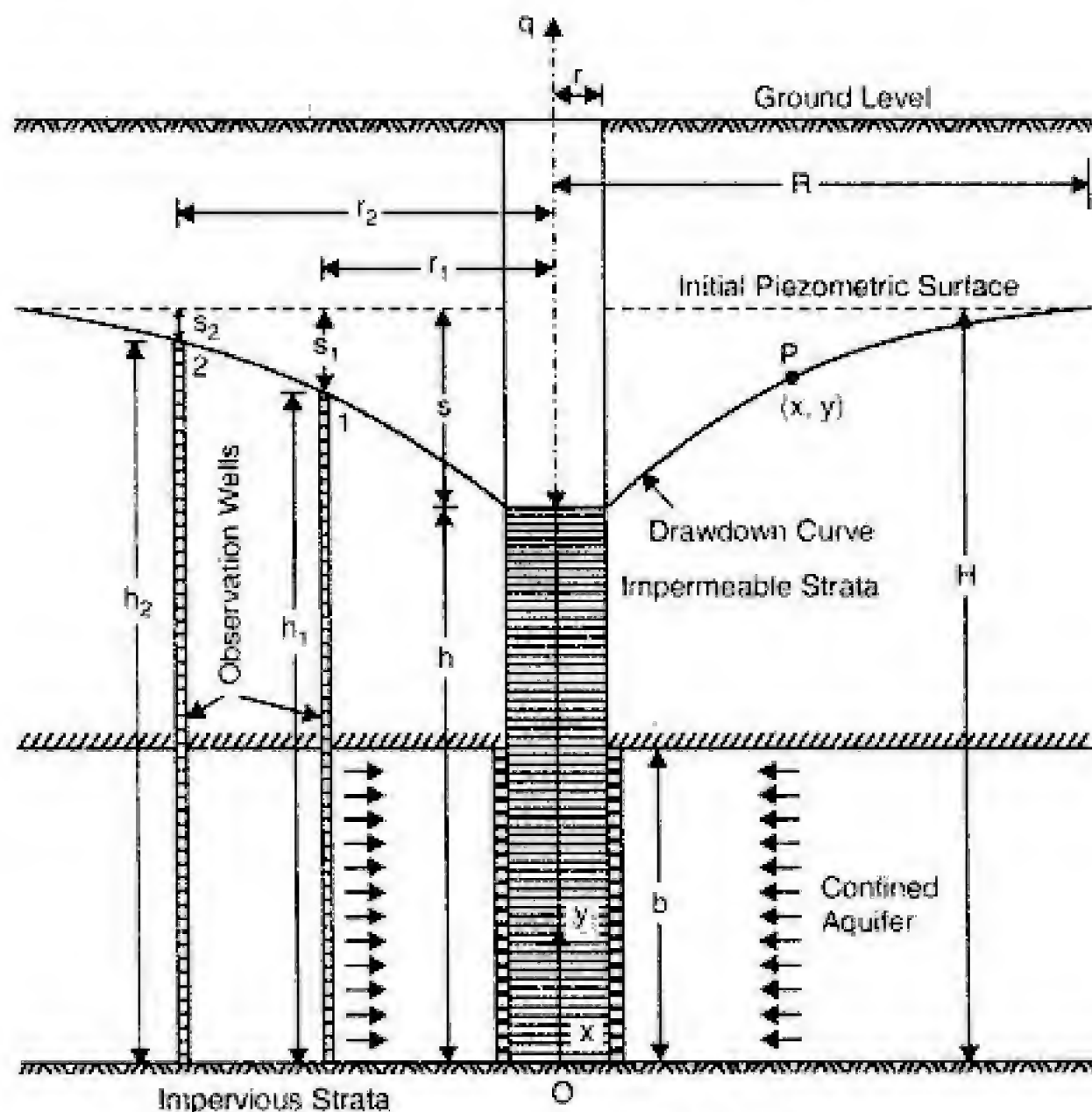


FIG. 8.3. CONFINED AQUIFER.



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sand layer is 20 m thick and its co-efficient of permeability is  $4 \times 10^{-3}$  cm/sec. Determine the rate of flow in  $\text{m}^3/\text{day}/\text{m}$  width of the aquifer.

**Solution :** The slope of the free water table represents the hydraulic gradient under which flow occurs. Let the co-ordinates of any point on the water table be measured with respect to the origin situated at the bottom of observation well No. 1. The co-ordinates of any point  $P$  are therefore  $(x, z)$ .

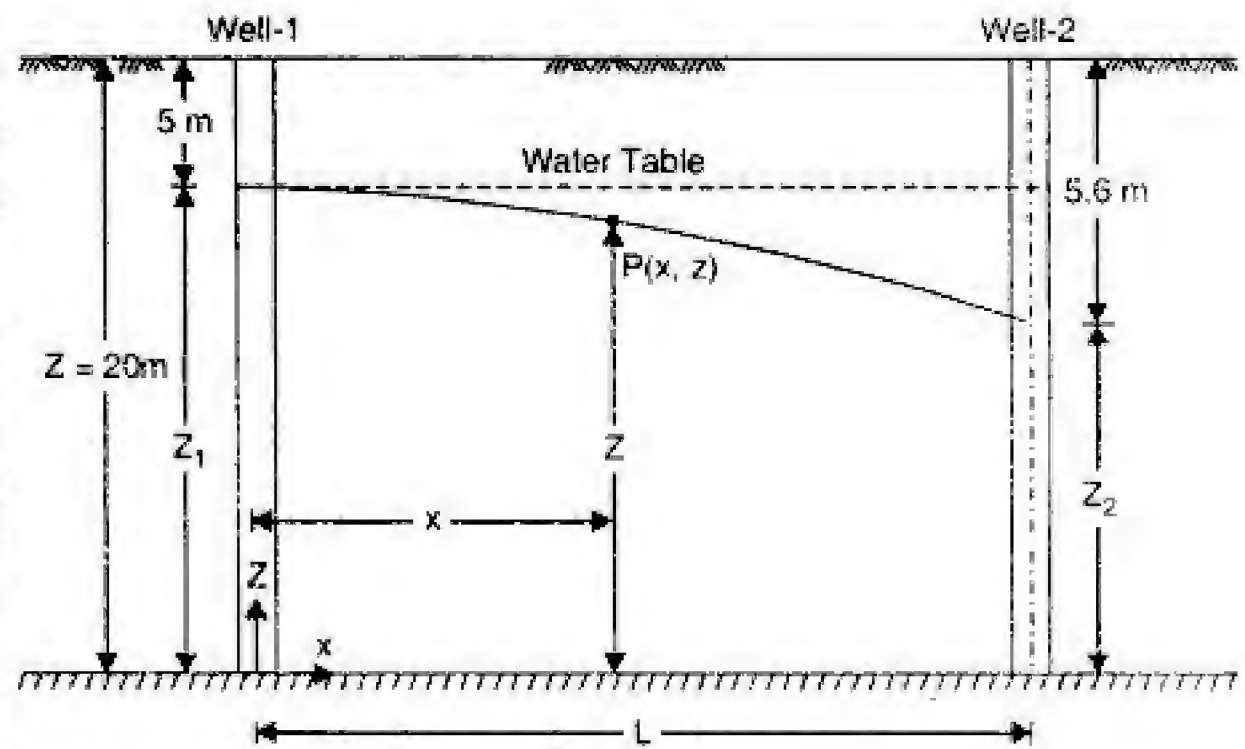


FIG. 8.5

$$i = -\frac{dz}{dx} \quad (\text{Dupuit's assumption})$$

$$\therefore q = kiA = -k \frac{dz}{dx} (z) \quad \text{per unit width of the aquifer}$$

$$\text{Integrating, } q \int_0^L dx = -k \int_{z_1}^{z_2} z dz \quad \therefore q = k \frac{z_1^2 - z_2^2}{2L} = k \left( \frac{z_1 - z_2}{L} \right) \left( \frac{z_1 + z_2}{2} \right)$$

$$\text{Now, } z_1 = 20 - 5 = 15 \quad ; \quad z_2 = 20 - 5.6 = 14.4 \quad \text{m} \quad ; \quad k = 4 \times 10^{-3} \text{ cm/sec} = 4 \times 10^{-5} \text{ m/sec}$$

$$\therefore q = 4 \times 10^{-5} \left( \frac{15 - 14.4}{290} \right) \times \left( \frac{15 + 14.4}{2} \right) = 1.215 \times 10^{-6} \text{ m}^3/\text{sec}/\text{m width}$$

$$= 1.215 \times 10^{-6} \times 24 \times 60 \times 60 \times \text{m}^3/\text{day}/\text{m width} = 0.105 \text{ m}^3/\text{day}/\text{m}$$

## 8.5. PUMPING IN TESTS

The U.S. Bureau of Reclamation (Earth Manual, 1960) has devised two types of pumping-in tests : (i) *Open-end tests* and (ii) *Packer tests*.

(i) **Open-end tests.** An open-end pipe is sunk in the strata and the soil is taken out of the pipe just to the bottom. Clean water, having temperature slightly higher than the ground water, is added through a metering system to maintain *gravity flow* under constant head. Water may also be allowed to enter the hole under some pressure head.

The permeability is calculated from the following expression determined from the electrical analogy experiments  $k = \frac{q}{5.5rh}$  ... (8.18)

where  $h$  = differential head of water (gravity plus pressure, if any)  
 $r$  = radius of casing ;  $q$  = constant rate of flow.

(ii) **Packer Tests.** An uncased portion of the drill hole or a perforated portion of the casing is used for performing the test. In case the test is performed during drilling,



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$$\therefore \frac{q_s}{q} = \frac{r}{b} \log_e \frac{R}{r} = 2.3 \frac{r}{b} \log_{10} \frac{R}{r} \quad \dots(8.27)$$

As a numerical example, let  $r = 8 \text{ cm} = 0.08 \text{ m}$ ;

$$\frac{R}{r} = 1000 ; b = 16 \text{ m} \quad \therefore \frac{q_s}{q} = 2.3 \times \frac{0.08}{16} \log_{10} 1000 \approx \frac{1}{30}$$

This shows that spherical flow is much less efficient than the radial flow.

### 8.10. FLOW TOWARDS OPEN WELL : RECUPERATION TEST

The hydraulics of flow towards open well is slightly different. In the case of tube well, radial flow takes place all around the well and there is no flow from the bottom of the well, while in the case of an open well, all the flow is essentially from the bottom. An open well has relatively larger diameter at its base, and its sides are mostly lined with brick. The discharge of an open well can be determined with the help of a *recuperation test*. In the recuperation test, water level is depressed to any level below the normal level, and the pumping is stopped. The time taken for the water to recuperate to the normal level is noted. From the data, the discharge from the well can be calculated as under:

Let  $aa$  = static water level in the well, before the pumping started

$bb$  = water level in the well when pumping stopped

$h_1$  = depression head when the pumping stopped (metres)

$cc$  = water level in the well at a time  $T$  after pumping stopped

$h_2$  = depression head in the well at a time  $T$  after pumping stopped (metres)

$h$  = depression head in the well at a time  $t$  after the pumping stopped (metres)

$dh$  = decrease in depression head  
in a time  $dt$

$t, T$  = time in hours

Thus, in a time  $t$ , reckoned from the instant of stopping the pumping, the water level recuperates by  $(h_1 - h)$  metres. In a time  $dt$  after this, the head recuperates by a value  $dh$  metres.

$\therefore$  Volume of water entering the well, when the head recuperates by  $dh$  is

$$dV = A dh \quad \dots(1)$$

where  $A$  = cross-sectional area of the well at its bottom.

Again, if  $q$  is the rate of discharge in the well at the time  $t$ , under the depression head  $h$ , the volume of water entering the well in a time  $dt$  hours is given by

$$dV = q dt ; \text{ But } q \propto h \text{ or } q = Kh$$

$$\therefore dV = Kh dt \quad \dots(2)$$

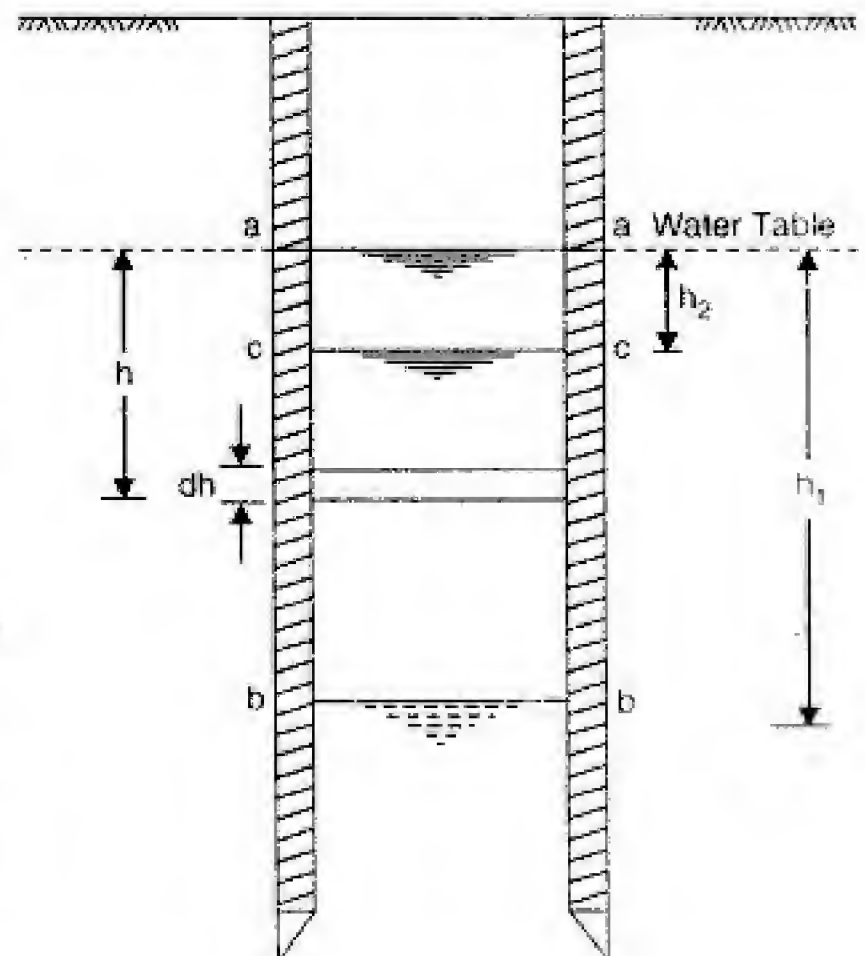


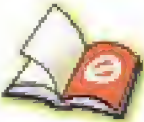
FIG. 8.10. RECUPERATION TEST



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### 8.11. EXAMPLES FROM COMPETITIVE EXAMINATIONS

**Example 8.7.** A 30 cm well completely penetrates an unconfined aquifer of depth 40 m. After a long period of pumping at a steady rate of 1500 lpm, the drawdown in two observation wells 25 m and 75 m from the pumping well were found to be 3.5 m and 2.0 m respectively. Determine the transmissibility of the aquifer. What is the drawn down at the pumping well ?

(Civil Services Exam. 1994)

**Solution :**

$$q = 1500 \text{ litres/min} = 1.5 \text{ m}^3/\text{min} = 0.025 \text{ m}^3/\text{sec}$$

$$h_2 = H - s_2 = 40 - 2 = 38 \text{ m} ; h_1 = H - s_1 = 40 - 3.5 = 36.5 \text{ m}$$

$$\text{From Eq. 8.7, } q = \frac{1.36 k (h_2^2 - h_1^2)}{\log_{10} r_2/r_1} \quad \text{or} \quad 0.025 = \frac{1.36 k (38^2 - 36.5^2)}{\log_{10} 75/25}$$

From which

$$k = 7.848 \times 10^{-5} \text{ m/sec}$$

$\therefore$

$$T = Hk = 40 \times 7.848 \times 10^{-5} = 3.14 \times 10^{-3} \text{ m}^2/\text{sec}$$

Also, at the well face,

$$q = \frac{1.36 k (h_1^2 - h_w^2)}{\log_{10} (r_1/r_w)} \quad \therefore \quad 0.025 = \frac{1.36 \times 7.848 \times 10^{-5} (36.5^2 - h_w^2)}{\log_{10} (25/0.15)}$$

or

$$h_w^2 = 36.5^2 - 520.42 = 811.83 \quad \text{From which } h_w = 28.49 \text{ m}$$

$\therefore$  Draw down at well face,  $s_w = H - h_w = 40 - 28.49 = 11.51 \text{ m}$

**Example 8.8.** During a recuperation test, the water in an open well was depressed by pumping by 2.5 m and it recuperated 1.8 m in 80 minutes. Calculate the yield from a well 4 m diameter under a depression head of 3 m.

(Engg. Services Exam. 1993)

**Solution :** This problem was set from example 8.4 of author's book. For solution, see example 8.4.

**Example 8.9.** An aquifer of 20 m average thickness is overlain by an impermeable layer of 30 m thickness. A test well of 0.5 m diameter and two observation wells at distances of 10 m and 60 m from the test well are drilled through the aquifer. After pumping at a rate of  $0.1 \text{ m}^3/\text{sec}$  for a long time, the following drawdowns are stabilized in these wells : First observation well, 4 m; second observation well, 3 m. Show the arrangement in a diagram. Determine the coefficient of permeability and drawdown in the test well.

(Engg. Services Exam. 1995)

**Solution :** Refer Eq. 8.11. Here  $h_2 = (20 + 30) - 3 = 47 \text{ m}$ ;  $h_1 = (20 + 30) - 4 = 46 \text{ m}$

$$r_2 = 60 \text{ m}; r_1 = 10 \text{ m}; b = 20 \text{ m}; q = 0.1 \text{ m}^3/\text{sec}.$$

$$k = \frac{q \log_{10} r_2/r_1}{2.72 b (h_2 - h_1)} = \frac{0.1 \log_{10} (60/10)}{2.72 \times 20 (47 - 46)} = 1.43 \times 10^{-3} \text{ m/sec}$$

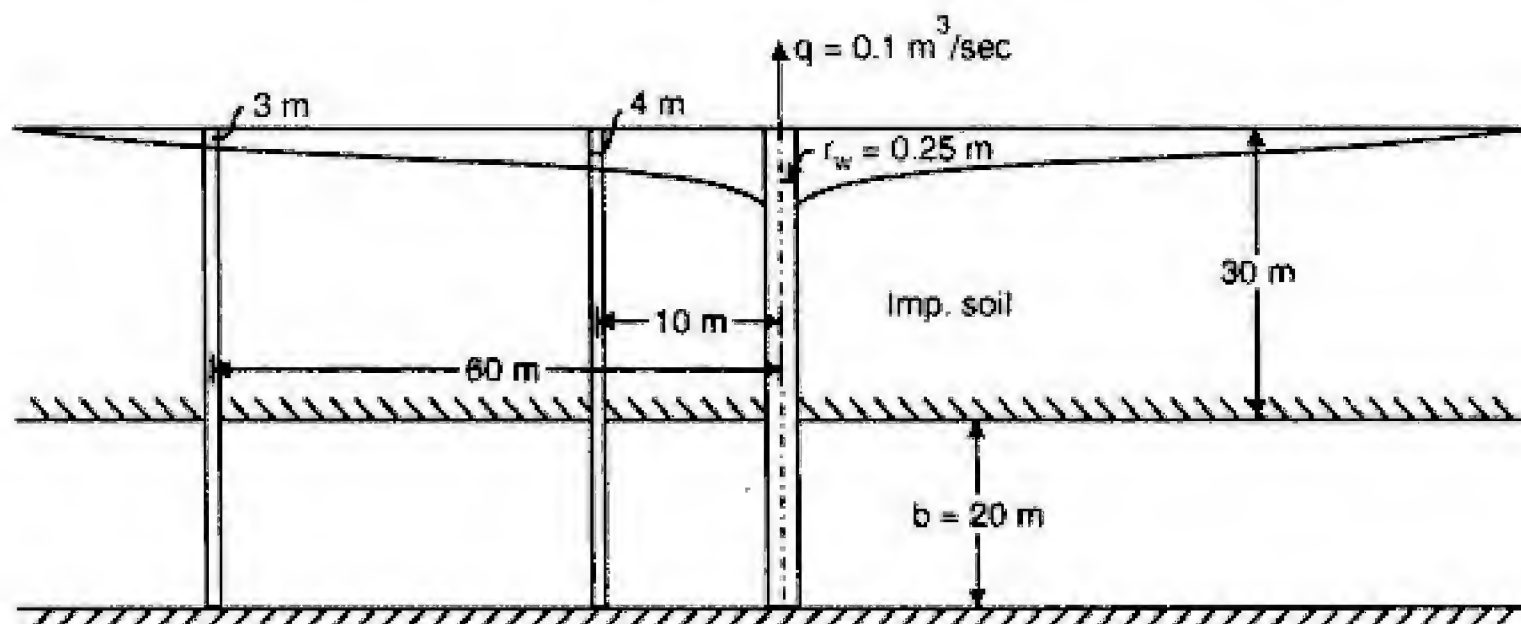


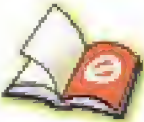
FIG. 8.11



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of flow. Thus, the effective pressure in a soil mass subjected to seepage pressure is given by

$$\sigma' = z\gamma' \pm p_s = z\gamma' \pm iz\gamma_w \quad \dots(9.7)$$

If the flow occurs in the *downward* direction, the effective pressure is *increased* and hence + sign is used. If, however, flow occurs in *upward* direction, the effective pressure is *decreased* and hence - sign is used in Eq. 9.7.

### 9.3. UPWARD FLOW : QUICK CONDITION

When flow takes place in an upward direction, the seepage pressure also acts in the upward direction and the effective pressure is reduced. If the seepage pressure becomes equal to the pressure due to submerged weight of the soil, the effective pressure is reduced to zero. In such a case, a cohesionless soil loses all its shear strength, and the soil particles have a tendency to move up in the direction of flow. This phenomenon of lifting of soil particles is called *quick condition*, *boiling condition* or *quick sand*. Thus, during the quick condition,

$$\sigma' = z\gamma' - p_s = 0$$

$$\text{or } p_s = z\gamma' \quad \text{or } iz\gamma_w = z\gamma'$$

from which

$$i = i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} \quad \dots(9.8)$$

The hydraulic gradient at such a critical state is called the *critical hydraulic gradient*. For loose deposits of sand or silt, if voids ratio  $e$  is taken as 0.67 and  $G$  as 2.67, the critical hydraulic gradient works out to be *unity*. It should be noted that *quick sand* is not a type of sand but a *flow condition* occurring within a cohesionless soil when its effective pressure is reduced to zero due to upward flow of water.

Fig. 9.2 shows a set-up to demonstrate the phenomenon of quick sand. Water flows in an upward direction through

a saturated soil sample of thickness  $z$  under a hydraulic head  $h$ . This head can be increased or decreased by moving the supply tank in the upward or downward direction. When the soil particles are in the state of critical equilibrium, the total upward force at the bottom of the soil becomes equal to the total weight of all the materials above the surface considered.

Equating the upward and downward forces at the level  $a-a$ , we have

$$(h + z) \gamma_w A = z\gamma_{sat} A \quad \therefore h\gamma_w = z(\gamma_{sat} - \gamma_w) = z\gamma'$$

$$\frac{h}{z} = i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e}$$

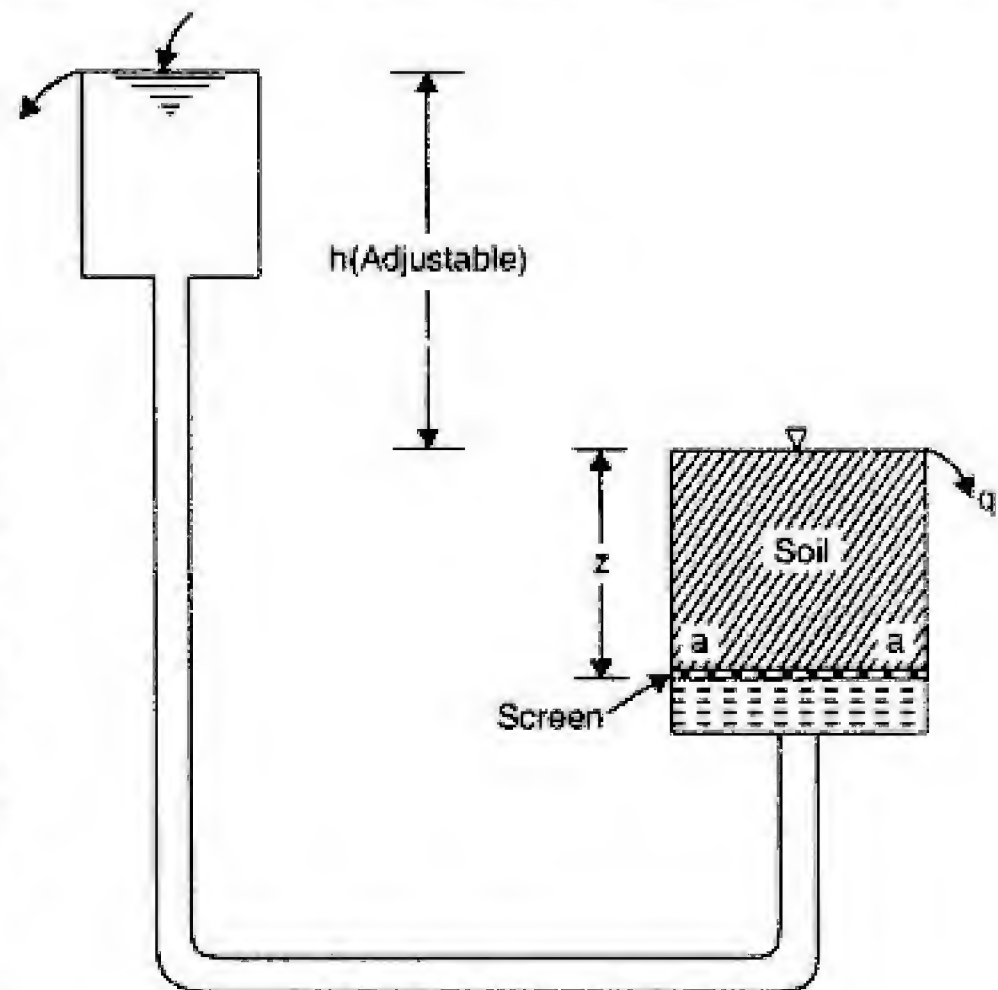


FIG. 9.2. QUICK SAND CONDITION.



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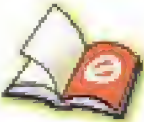


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$$k' \frac{\Delta h}{l} (I) = k_x \frac{\Delta h}{l \sqrt{k_x/k_y}} (I) \quad \therefore \quad k' = k_x \sqrt{\frac{k_y}{k_x}} = \sqrt{k_x k_y} \quad \dots(9.25)$$

$$\text{Hence the discharge is given by} \quad q = k' H \frac{N_f}{N_d} = \sqrt{k_x \cdot k_y} H \frac{N_f}{N_d} \quad \dots(9.26)$$

### 9.9. DEFLECTION OF FLOW LINES AT INTERFACE OF DISSIMILAR SOILS

The flow lines get deflected at the interface between two dissimilar soils when they pass from one soil to the other. Fig. 9.14 shows two flow lines  $\psi_1$  (AB) and  $\psi_2$  (CD) being deflected to  $BA_1$  and  $DC_1$  respectively at the interface of two soils having permeability coefficients  $k_1, k_2$ . Let  $\phi_1$  and  $\phi_2$  be two equipotential lines, having a drop of potential  $\Delta h$  between them. The equipotential lines also get deflected at the interface. Let  $\theta_1$  and  $\theta_2$  be the angles that the flow line  $\psi_1$  makes with the normal to the interface before and after deflection respectively.

The rate of flow  $\Delta q$  between the flow channel bounded by  $\psi_1, \psi_2$  is given by

$$\Delta q = k_1 i_1 (ED) = k_2 i_2 (BF)$$

$$\text{where} \quad i_1 = \frac{\Delta h}{EB} \quad \text{and} \quad i_2 = \frac{\Delta h}{DF}$$

$$\therefore \quad \Delta q = k_1 \frac{\Delta h}{EB} ED = k_2 \frac{\Delta h}{DF} BF$$

$$\text{or} \quad \frac{k_1}{k_2} = \frac{EB}{ED} \frac{BF}{DF} ; \quad \text{But} \quad \frac{EB}{ED} = \tan \theta_1$$

$$\text{and} \quad \frac{DF}{BF} = \tan \theta_2$$

$$\therefore \quad \frac{\tan \theta_1}{\tan \theta_2} = \frac{k_1}{k_2} \quad \dots(9.27)$$

Eq. 9.27 defines the law of deflection of flow line at the interface of two dissimilar soils.

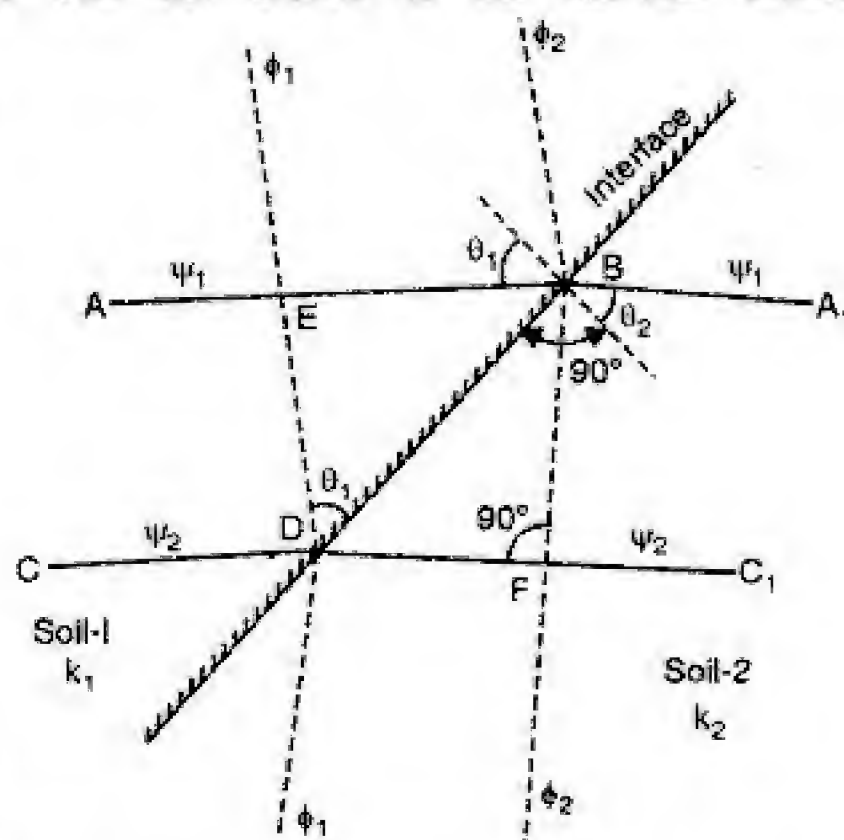


FIG. 9.14. DEFLECTION OF FLOW LINES

### 9.10. PHREATIC LINE OF AN EARTH DAM

Let us now consider the case of a homogeneous earth dam with a horizontal filter, as shown in Fig. 9.15(b). In order to draw the flow net, it is first essential to find the location and shape of the *phreatic line* or the *top flow line* separating the saturated and unsaturated zones. The phreatic line or seepage line is defined as the line within a dam section below which there are positive hydrostatic pressures in the dam. The hydrostatic pressure on the phreatic line itself is atmospheric. The phreatic line can be located by (i) analytical method, (ii) graphical method, and (iii) experimental method. We shall first discuss the graphical method of determination of phreatic line suggested by *Casagrande*.

**Casagrande Method.** An analytically derived flow net has been given by Kozeny (1931) for the case of water flowing above an impervious, infinite, horizontal plane which



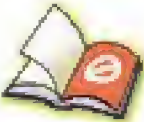
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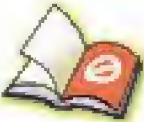




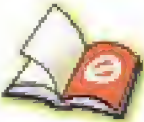
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**Step 7 : Pressure distribution for Length AE**

In this length,  $y = 0$ . Also, introducing a new variable  $\beta_1'$  defined by the equation,

$$\lambda \cos v - \lambda_1 = -\cosh \beta_1' \quad \dots(x)$$

we get, from Eq. 10.12,

$$x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' - \frac{d_1 - d_2}{\pi} \log_e [-\cosh \beta_1' + \sinh \beta_1'] + i(d_1 + d_2)$$

$$\text{or } x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' + \frac{d_1 - d_2}{\pi} \beta_1' - \frac{d_1 - d_2}{\pi} \log_e (-1) + i(d_1 - d_2)$$

$$x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' + \frac{d_1 - d_2}{\pi} \beta_1'$$

$$\text{Hence } \frac{x}{d_1 - d_2} \pi K = \delta_x \pi K = \sinh \beta_1' + K \beta_1' \quad \dots(xi)$$

From this equation,  $\beta_1'$  can be calculated. Hence from Eq (x), the pressure function  $v$  is found to be

$$v = \cos^{-1} \left( \frac{\lambda_1 - \cosh \beta_1'}{\lambda} \right) = \pi \frac{P}{H}$$

$$\therefore P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad \dots(10.13 \ a)$$

$$\text{or } \phi = \frac{P}{H} = \frac{1}{\pi} \cos^{-1} \frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad \dots(10.13 \ b)$$

This gives pressure distribution under AE.

**Step 8 : Pressure distribution for Length CB :**

Introducing a new variable  $\beta_2'$  defined by the equation  $\lambda \cos v - \lambda_1 = \cosh \beta_2'$ , we get from Eq. 10.12,

$$\sinh \beta_2' - K \beta_2' = \pi K \frac{x}{d_1 - d_2} = \delta_x \pi K \quad \dots(xii)$$

From this equation,  $\beta_2'$  can be known. Hence the pressure distribution is given by

$$v = \cos^{-1} \frac{\lambda_1 + \cosh \beta_2'}{\lambda} = \pi \frac{P}{H}$$

$$\text{Hence } P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + \cosh \beta_2'}{\lambda} \quad \dots(10.14 \ a)$$

$$\text{or } \phi = \frac{1}{\pi} \cos^{-1} \left( \frac{\lambda_1 + \cosh \beta_2'}{\lambda} \right) \quad \dots(10.14 \ b)$$

Combining Eqs. 10.13 and 10.14 in one common form, we get

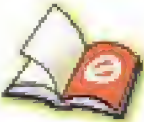
$$P = \frac{H}{\pi} \cos^{-1} \left( \frac{\lambda_1 \pm \cosh \beta_{1,2}'}{\lambda} \right) \quad \dots(10.15)$$

where - sign is used for upstream and + sign for the downstream part of the floor.



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Type of soil	Exit gradient ( $G_E$ )
1. Fine sand	$\frac{1}{6}$ to $\frac{1}{7}$
2. Coarse sand	$\frac{1}{5}$ to $\frac{1}{6}$
3. Shingle	$\frac{1}{4}$ to $\frac{1}{5}$

### 10.5. PAVLOVSKY'S METHOD : FINITE DEPTH PROBLEMS

Pavlovsky analysed various cases of seepage under the hydraulic structures in 1922, but as the text was in Russian, his work remained almost unknown. Apart from a purely mathematical analysis, Pavlovsky's investigations also comprised model-tank tests as well as the electrical analogy models. His analysis includes the problems of seepage under the aprons founded both finite as well as infinite depth. The transformations are performed into two operations : (i) in the first operation the field of the true profile ( $z$ -plane) is transformed on to the auxiliary semi-infinite plane ( $\zeta$ -plane) and (ii) in the second operation, a rectangular field of known flow characteristics ( $Z$ -plane) is transformed on the the same semi-infinite plane ( $\zeta$ -plane). Thus, from the first operation, we get a transformation equation between  $z$  and  $\zeta$ , while in the second operation, we get a transformation equation between  $Z$  and  $\zeta$ . Finally, combining the two operations, we get a transformation equation between  $z$  and  $Z$ , which is the main purpose of the two stage transformation.

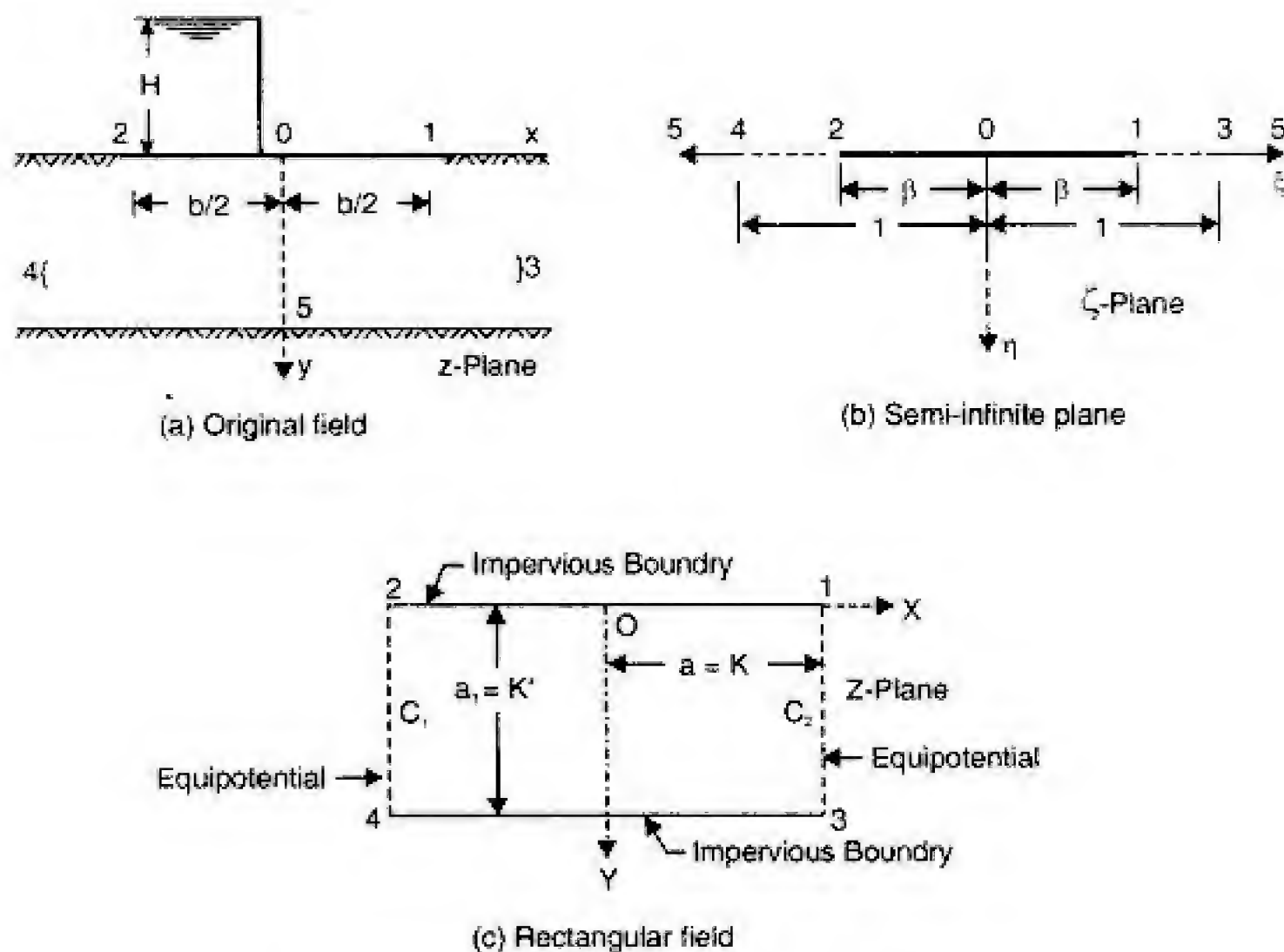


FIG. 10.7. PAVLOVSKY'S TRANSFORMATIONS.



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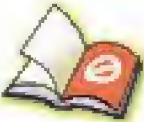




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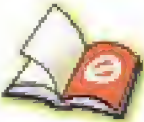
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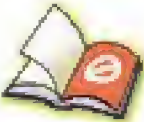
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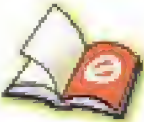


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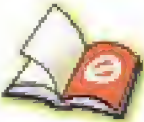
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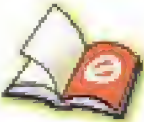


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which on simplification reduces to :  $\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + Z = 0 \quad \dots(\text{III}) \quad \dots(12.3 \text{ c})$

Eq. 12.3 a, b, c are three equilibrium equations which are also sometimes written in the following order :

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} + X = 0 \quad \dots(\text{I})$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} + Y = 0 \quad \dots(\text{II}) \dots(12.3)$$

$$\frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + Z = 0 \quad \dots(\text{III})$$

However, we have seen in §12.1. that there are six independent stress components acting at a point and the complete solution of the problem requires the determination of these six stress components. Thus there are six unknowns, and only three *equations of equilibrium* are available. Thus, the problem of elasticity is strictly of indeterminate nature. These equations of static equilibrium must be supplemented with equations of compatibility (§12.5) of deformations to get the complete solution. In addition to this, the final solution should satisfy the boundary conditions (§ 12.6).

### 12.3. EQUILIBRIUM EQUATIONS FOR SATURATED SOIL BODY

The equilibrium equations derived in the previous article are satisfied both in terms of the total stresses and the effective stresses acting on a saturated soil element in equilibrium.

(a) **Total stresses.** If the equilibrium equations are formulated in terms of total stresses which include the water pressure or seepage force, the body forces will be equal to those due to gravity in the respective directions. If z-axis is directed downwards, we have :  $X=0$ ,  $Y=0$  and  $Z=\gamma$ . Thus, the equilibrium equations in terms of total stresses can be written as :

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} = 0 \quad \dots(12.4 \text{ a})$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} = 0 \quad \dots(12.4 \text{ b})$$

and  $\frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + \gamma = 0 \quad \dots(12.4 \text{ c})$

where  $\sigma_x$ ,  $\sigma_y$  and  $\sigma_z$  are the total stresses, which include the seepage forces. Thus, Eqs. 12.4 are the equilibrium equations in terms of the total stresses.

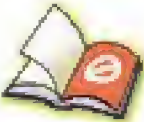
Now  $\sigma_x = \sigma_x' + u = \sigma_x' + \gamma_w (h - h_e)$

$u$  = pore pressure =  $\gamma_w h_w = \gamma_w (h - h_e)$  ;  $h$  = total head at the point

$h_e$  = elevation head, which is a function of  $z$

Differentiating (i) partially with respect to  $x$ , we get

$$\therefore \frac{\partial \sigma_x}{\partial x} = \frac{\partial \sigma_x'}{\partial x} + \gamma_w \frac{\partial h}{\partial x} \quad \left[ \text{since } \frac{\partial h_e}{\partial x} = 0 \right]$$



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Similarly, 
$$2 \frac{\partial^2 \epsilon_y}{\partial z \partial x} = \frac{\partial}{\partial x} \left( \frac{\partial Y_{yz}}{\partial x} - \frac{\partial Y_{zx}}{\partial y} + \frac{\partial Y_{xy}}{\partial z} \right) \quad \dots(12.10 \ e)$$

and 
$$2 \frac{\partial^2 \epsilon_z}{\partial x \partial y} = \frac{\partial}{\partial z} \left( \frac{\partial Y_{yz}}{\partial x} + \frac{\partial Y_{zx}}{\partial y} - \frac{\partial Y_{xy}}{\partial z} \right) \quad \dots(12.10 \ f)$$

Eqs. 12.10 are the compatibility equations.

## 12.6. BOUNDARY CONDITION EQUATIONS

The complete solution of an elasticity problem is obtained by the solution of equilibrium and compatibility equations, but the final solution must also satisfy the boundary condition equations. In order to derive the boundary condition equations, consider a boundary plane  $ABC$  (Fig.12.4 a) with  $l$ ,  $m$  and  $n$  as the direction cosines of the external normal to its surface at any point. Let  $\bar{X}$ ,  $\bar{Y}$  and  $\bar{Z}$  be the components of surface forces per unit area on the elementary area  $ABC$ . Fig. 12.4 a shows the nine stress components on the faces  $OBC$ ,  $OAC$  and  $OAB$ . If the element volume is considered to be shrunk to a point, these nine stress components are assumed to act at the point.

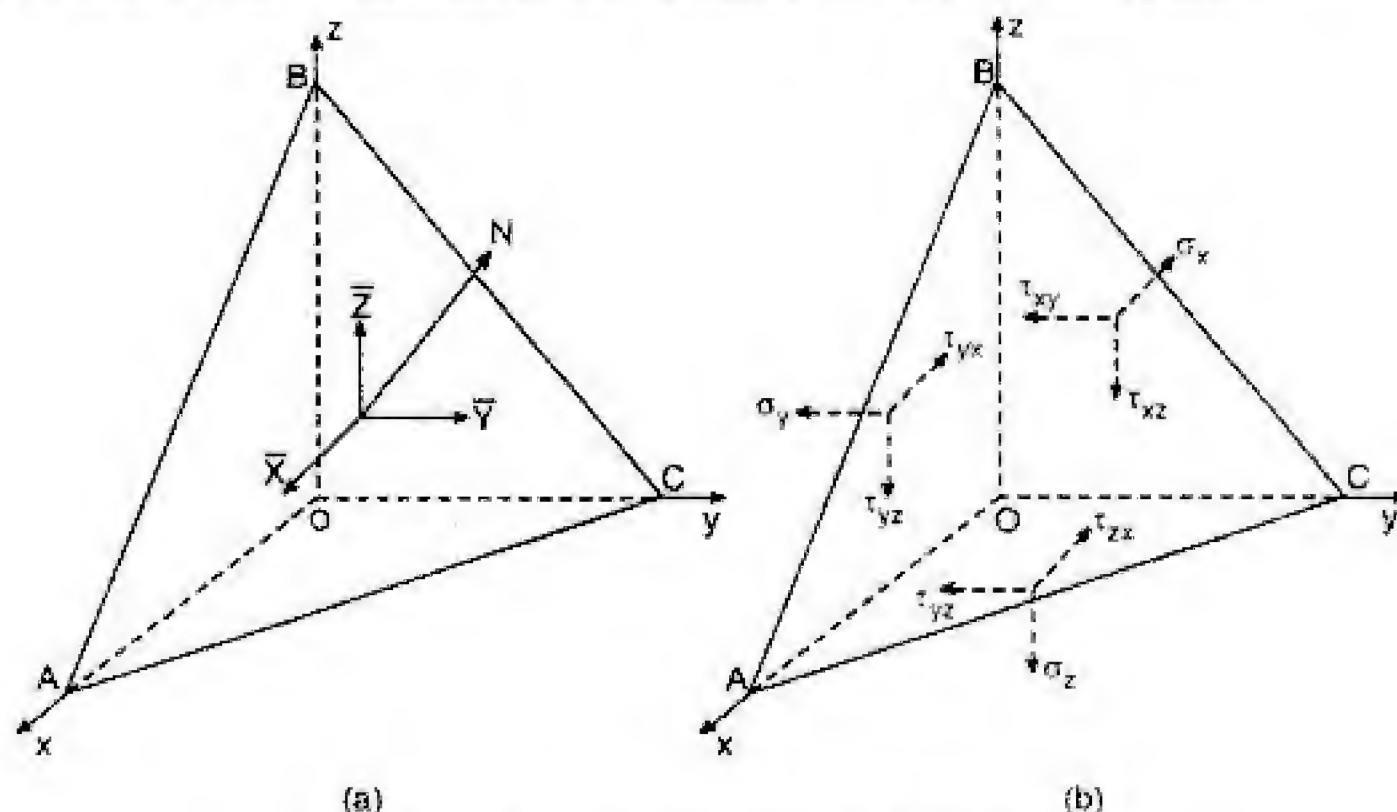


FIG. 12.4. BOUNDARY CONDITIONS.

Let area  $ABC = ds$   $\therefore$  Area  $OBC = ds \cos (N, x) = ds \cdot l$

$OAB = ds \cos (N, y) = ds \cdot m$  ;  $OAC = ds \cos (N, z) = ds \cdot n$

Resolving all the forces in  $x$ -direction, and equating them to zero, we get

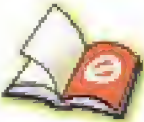
$$\Sigma X = 0 = \bar{X}ds - \sigma_x ds l - \tau_{yx} ds m - \tau_{zx} ds n$$

or 
$$\bar{X} = \sigma_x l + \tau_{yx} m + \tau_{zx} n \quad \dots(12.11 \ a)$$

Similarly, 
$$\bar{Y} = \sigma_y m + \tau_{zy} n + \tau_{xy} l \quad \dots(12.11 \ b)$$

and 
$$\bar{Z} = \sigma_z n + \tau_{xz} l + \tau_{yz} m \quad \dots(12.11 \ c)$$

These are the *boundary condition equations*, which are also sometimes written in the matrix form :



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Substituting the value of  $\frac{2\partial^2\tau_{xz}}{\partial x\partial z}$  in (i), we get

$$\frac{\partial^2}{\partial z^2}(\sigma_x - \mu\sigma_z) + \frac{\partial^2}{\partial x^2}(\sigma_z - \mu\sigma_x) + (1 + \mu)\left(\frac{\partial^2\sigma_x}{\partial x^2} + \frac{\partial^2\sigma_z}{\partial z^2}\right) + (1 + \mu)\left(\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right) = 0$$

This, on simplification, gives

$$\frac{\partial^2\sigma_x}{\partial z^2} + \frac{\partial^2\sigma_z}{\partial x^2} + \frac{\partial^2\sigma_x}{\partial x^2} + \frac{\partial^2\sigma_z}{\partial z^2} = -(1 + \mu)\left(\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right)$$

$$\text{or} \quad \left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial z^2}\right)(\sigma_x + \sigma_z) = -(1 + \mu)\left(\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right) \quad \dots(12.20)$$

If body forces are absent, or constant, we have

$$\left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial z^2}\right)(\sigma_x + \sigma_z) = 0 \quad \dots(12.21)$$

which is the required compatibility equation in terms of stress, for the plane stress case.

## (2) Plane strain case

Substituting the values of  $\epsilon_x$ ,  $\epsilon_y$  and  $\gamma_{xz}$  (Eq.12.17), we get from Eq.12.19,

$$(1 - \mu^2)\frac{\partial^2\sigma_x}{\partial z^2} - \mu(1 + \mu)\frac{\partial^2\sigma_z}{\partial z^2} + (1 - \mu^2)\frac{\partial^2\sigma_z}{\partial x^2} - \mu(1 + \mu)\frac{\partial^2\sigma_x}{\partial x^2} = 2(1 + \mu)\frac{\partial^2\tau_{xz}}{\partial x\partial z} \quad \dots(iii)$$

Substituting the value of  $\frac{2\partial^2\tau_{xz}}{\partial x\partial z}$  as found in (ii), we get

$$(1 - \mu)^2\frac{\partial^2\sigma_x}{\partial z^2} - \mu(1 + \mu)\frac{\partial^2\sigma_x}{\partial z^2} + (1 - \mu^2)\frac{\partial^2\sigma_z}{\partial x^2} - \mu(1 + \mu)\frac{\partial^2\sigma_x}{\partial x^2} + (1 + \mu)\left[\frac{\partial^2\sigma_x}{\partial x^2} + \frac{\partial^2\sigma_z}{\partial z^2} + \frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right] = 0$$

which on simplification, reduces to ,

$$\frac{\partial^2\sigma_x}{\partial z^2} + \frac{\partial^2\sigma_z}{\partial z^2} + \frac{\partial^2\sigma_z}{\partial x^2} + \frac{\partial^2\sigma_x}{\partial x^2} = -\frac{1}{1 - \mu}\left(\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right)$$

$$\text{or} \quad \left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial z^2}\right)(\sigma_x + \sigma_z) = -\frac{1}{1 - \mu}\left(\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial z}\right) \quad \dots(12.22)$$

If the body forces are absent, or constant, we have

$$\left(\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial z^2}\right)(\sigma_x + \sigma_z) = 0 \quad \dots(12.23)$$

which is the same as Eq. 12.21 found for the plane stress case. Thus, in case of constant body forces (or no body forces), same compatibility equation holds both for the case of plane stress and for case of plane strain. Hence the stress distribution is the same in both the cases, provided the shape of the boundary and the external forces are the same. Also, the stress distribution is the same for all the isotropic materials, since Eq.12.21 or 12.23 do not contain any elastic constant. The photo-elastic method of determination of stress distribution is based on this conclusion.



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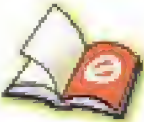


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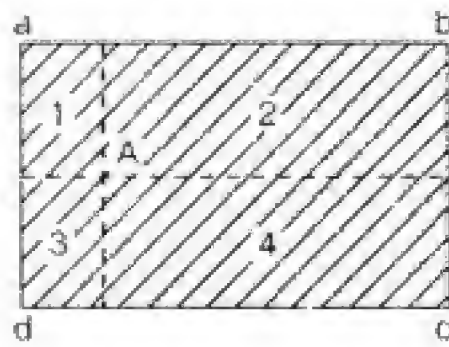


FIG. 13.11. POINT A INSIDE THE RECTANGLE.

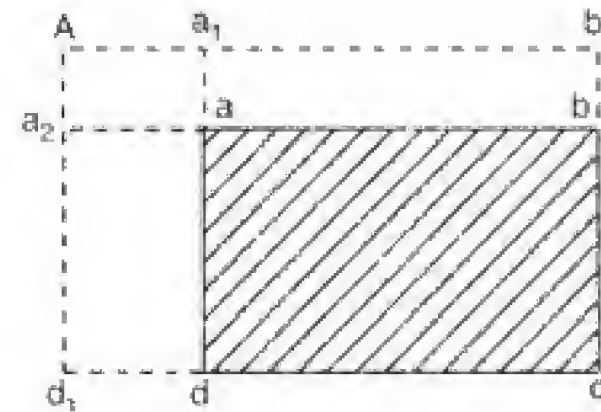


FIG. 13.12. POINT A OUTSIDE THE RECTANGLE.

$$\sigma_z = q(K_1 + K_2 + K_3 + K_4) \quad \dots(13.25)$$

where  $K_1$ ,  $K_2$ ,  $K_3$  and  $K_4$  are the influence factors for the four rectangles 1, 2, 3 and 4. Similarly, if the point A is *outside* the loaded rectangle, construct the four rectangles as shown in Fig.13.12. The shaded area in the loaded rectangle may be considered to be the algebraic sum of the four rectangles, each with the corner at A ;

$$\text{Area } abcd = Ab_1cd_1 - Ab_1ba_2 - Aa_1dd_1 + Aa_1aa_2$$

$$\therefore \sigma_z \text{ at } A = q(K_1 - K_2 - K_3 + K_4) \quad \dots(13.26)$$

where  $K_1$  = influence factor for area  $Ab_1cd_1$  ;  $K_2$  = influence factor for area  $Ab_1ba_2$   
 $K_3$  = influence factor for area  $Aa_1dd_1$  ;  $K_4$  = influence factor for area  $Aa_1aa_2$

TABLE 13.7. INFLUENCE FACTORS FOR VERTICAL PRESSURE UNDER A CORNER OF UNIFORMLY LOADED RECTANGULAR AREA (AFTER NEWMARK, 1935)

m	n									
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.1	0.0047	0.0092	0.0132	0.0168	0.0198	0.0222	0.0242	0.0258	0.0270	0.0279
0.2	0.0092	0.0179	0.0259	0.0328	0.0387	0.0435	0.0474	0.0504	0.0528	0.0547
0.3	0.0132	0.0259	0.0374	0.0474	0.0559	0.0629	0.0686	0.0731	0.0766	0.0794
0.4	0.0168	0.0328	0.0474	0.0602	0.0711	0.0801	0.0873	0.0931	0.0977	0.1013
0.5	0.0198	0.0387	0.0559	0.0711	0.0840	0.0947	0.1034	0.1104	0.1158	0.1202
0.6	0.0222	0.0435	0.0629	0.0801	0.0947	0.1069	0.1168	0.1247	0.1311	0.1391
0.7	0.0242	0.0474	0.0686	0.0873	0.1034	0.1168	0.1277	0.1365	0.1436	0.1491
0.8	0.0258	0.0504	0.0731	0.0931	0.1104	0.1247	0.1365	0.1461	0.1537	0.1598
0.9	0.0270	0.0528	0.0766	0.0977	0.1158	0.1311	0.1436	0.1537	0.1619	0.1684
1.0	0.0279	0.0547	0.0794	0.1013	0.1202	0.1361	0.1491	0.1598	0.1648	0.1752
1.2	0.0293	0.0573	0.0832	0.1063	0.1263	0.1431	0.1570	0.1684	0.1777	0.1851
1.4	0.0301	0.0589	0.0856	0.1094	0.1300	0.1475	0.1620	0.1739	0.1836	0.1914
1.6	0.0306	0.0599	0.0871	0.1114	0.1324	0.1503	0.1652	0.1774	0.1874	0.1955
1.8	0.0309	0.0606	0.0880	0.1126	0.1340	0.1521	0.1672	0.1797	0.1899	0.1981
2.0	0.0311	0.0610	0.0887	0.1134	0.1350	0.1533	0.1686	0.1812	0.1915	0.1999
2.5	0.0314	0.0616	0.0895	0.1145	0.1363	0.1548	0.1704	0.1832	0.1938	0.2024
3.0	0.0315	0.0618	0.0898	0.1150	0.1368	0.1555	0.1711	0.1841	0.1947	0.2034
5.0	0.0316	0.0620	0.0901	0.1154	0.1374	0.1561	0.1719	0.1849	0.1956	0.2046
10.0	0.0316	0.0620	0.0902	0.1154	0.1375	0.1562	0.1720	0.1850	0.1958	0.2044
$\infty$	0.0316	0.0620	0.0902	0.1154	0.1375	0.1562	0.1720	0.1850	0.1958	0.2046





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20 equal area units, each area unit will exert a pressure equal to  $0.005 q$  intensity at a depth of 5 cm.

Let the radius of second concentric circle be equal to  $r_2$  cm. By extending the twenty radial lines, the space between the two concentric circles is again divided into 20 equal area units;  $A_1 A_2 B_2 B_1$  is one such area unit. The vertical pressure, at the centre, due to each of these area units is to be of intensity  $0.005 q$ . Therefore, the total pressure due to area units  $O A_1 B_1$  and  $A_1 A_2 B_2 B_1$  at depth  $z = 5$  cm below the centre is  $2 \times 0.005 q$ . Hence from Eq. 13.15 :

$$\text{Vertical pressure due to } O A_2 B_2 = \frac{q}{20} \left[ 1 - \left\{ \frac{1}{1 + \left( \frac{r_2}{z} \right)^2} \right\}^{3/2} \right] = 2 \times 0.005 q$$

Substituting  $z = 5$  cm, we get  $r_2 = 2.00$  cm from the above relation. Similarly, the radii of 3rd, 4th 5th 6th 7th 8th, 9th circles can be calculated, as tabulated in Table 13.8. The radius of 10th circle is given by the following governing equation :

$$\frac{q}{20} \left[ 1 - \left\{ \frac{1}{1 + \left( \frac{r_{10}}{z} \right)^2} \right\}^{3/2} \right] = 10 \times 0.005 q = \frac{q}{20}$$

From the above  $r_{10} = \text{infinity}$ .

**TABLE 13.8 RADII OF CONCENTRIC CIRCLES FOR INFLUENCE CHART**  
( $z=5$  cm;  $q = 0.005$ ; each circle divided into 20 parts)

Number of circles	1	2	3	4	5	6	7	8	9	$9\frac{1}{2}$	10
Radius (cm)	1.35	2.00	2.59	3.18	3.83	4.59	5.54	6.94	9.54	12.62	$\infty$

Fig. 13.15. shows the influence chart drawn on the basis of Table 13.8.

To use the chart for determining the vertical stress at any point under the loaded area, the plan of the loaded area is first drawn on a tracing paper to such a scale that the length  $AB$  ( $=5$  cm) drawn on the chart represents the depth to the point at which pressure is required. For example, if the pressure is to be found at a depth of 5 m, the scale of plan will be  $5 \text{ cm} = 5 \text{ m}$ , or  $1 \text{ cm} = 1 \text{ m}$ . The plan of the loaded area is then so placed over the chart that the point below which pressure is required coincides with the centre of the chart. The point below which pressure is required may lie within or outside the loaded area. The total number of area units (including the fractions) covered by the plan of the loaded area is counted. The vertical pressure is then calculated from the relation :

$$\sigma_A = 0.005 q \times N_A \quad (\text{where } N_A = \text{number of area units under the loaded area}). \quad \dots(13.31)$$

**Example 13.3.** A rectangular area  $2 \text{ m} \times 4 \text{ m}$  carries a uniform load of  $80 \text{ kN/m}^2$  at the ground surface. Find the vertical pressures at 5 m below the centre and corner of the loaded area.

**Solution.** (a) For the point under the centre of the area, there will be influence of four rectangles of size  $1 \text{ m} \times 2 \text{ m}$ , having a common corner at the centre of the loaded rectangle.



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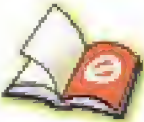
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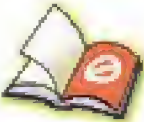
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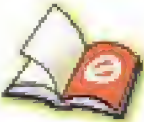


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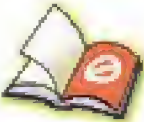
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For a given difference in pressure, the value of coefficient of compressibility decreases as the pressure increases.

**Coefficient of volume change  $m_v$ .** The coefficient of volume change or the coefficient of volume compressibility is defined as the change in volume of a soil per unit of initial volume due to a given unit increase in the pressure :

$$m_v = - \frac{\Delta e}{1 + e_0} \frac{1}{\Delta \sigma'} \quad \dots(15.11)$$

$$\text{Substituting } - \frac{\Delta e}{\Delta \sigma'} = a_v, \text{ we get } m_v = \frac{a_v}{1 + e_0} \quad \dots(15.12)$$

When the soil is laterally confined, the change in the volume is proportional to change in the thickness  $\Delta H$  and the initial volume is proportional to the initial thickness  $H_0$

$$\text{Hence } m_v = - \frac{\Delta H}{H_0} \frac{1}{\Delta \sigma'} \quad \dots(15.13)$$

Thus the change in the thickness,  $\Delta H$  due to pressure increment is given by

$$\Delta H = - m_v H_0 \Delta \sigma' \quad \dots(15.13 \text{ a})$$

(The minus sign in the above equation simply denotes that the voids ratio or thickness decreases with the increase in the pressure.)

### CONSOLIDATION SETTLEMENT.

Consolidations settlement can be computed by two methods :

Method (a) : Using coefficient of volume change ( $m_v$ )

Method (b) : Using voids ratio.

(a) **Final settlement using coefficient of volume change ( $m_v$ )**

The consolidation settlement  $\rho_f$  ( $\downarrow$ ) when the soil stratum of thickness  $H$  has fully consolidated under a pressure increment  $\Delta \sigma'$ , is given by Eq. 15.13 a :

$$\rho_f = m_v H \Delta \sigma' \quad \dots(15.14)$$

This is on the assumption that the pressure increment is transmitted uniformly over the thickness  $H$ . However, in practical cases under a finite surface loading, the intensity of  $\Delta \sigma'$  decreases with the depth of layer in a non-linear manner. In such circumstances, the consolidation settlement  $\Delta \rho_f$  of an element of thickness  $dz$  is calculated under an average effective pressure increment  $\Delta \sigma'$  by Eq. 15.14 :

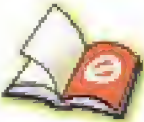
$$\Delta \rho_f = m_v \Delta \sigma' dz$$

Integrating for the total thickness  $H$  of the layer :

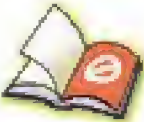
$$\rho_f = \int_0^H m_v \Delta \sigma' dz \quad \dots(15.14 \text{ a})$$

in which both  $m_v$  and  $\Delta \sigma'$  are variables. The integration may be performed numerically or graphically.

The numerical integration may be performed by dividing the total thickness  $H$  into a number of thin layers, and  $\Delta \sigma'$  at the mid-height of each layer may be considered to



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The quantity of water entering the soil element  $= v \, dx \, dy$

The quantity of water leaving the soil element  $= \left( v + \frac{\partial v}{\partial z} dz \right) dx \, dy$ .

Hence the net quantity of water  $dq$  squeezed out of the soil element per unit time is given by

$$\Delta q = \frac{\partial v}{\partial z} dx \, dy \, dz \quad \dots(v)$$

The decrease in the volume of soil is equal to the volume of water squeezed out. However, from Eq. 15.4.  $\Delta V = -m_v V_0 \Delta \sigma'$  ...(vi)

where  $V_0$  = volume of soil element at time  $t_0 = dx \, dy \, dz$ .

$\therefore$  Change of volume per unit time is given by

$$\frac{\partial(\Delta V)}{\partial t} = -m_v dx \, dy \, dz \frac{\partial(\Delta \sigma')}{\partial t} \quad \dots(vii)$$

Equating (v) and (vii), we get  $\frac{\partial v}{\partial z} = -m_v \frac{\partial(\Delta \sigma')}{\partial t}$  ...(viii)

Now  $\Delta \sigma = \Delta \sigma' + \bar{u}$ , where  $\Delta \sigma$  is constant.

$$\therefore \frac{\partial(\Delta \sigma')}{\partial t} = -\frac{\partial \bar{u}}{\partial t} \quad \dots(ix)$$

Hence, from (viii) and (ix),  $\frac{\partial v}{\partial z} = m_v \frac{\partial \bar{u}}{\partial t}$  ...(x)

Combining Eqs. (iv) and (x), we get

$$\frac{\partial \bar{u}}{\partial t} = \frac{k}{m_v \gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2} \quad \dots(15.16)$$

or 
$$\frac{\partial \bar{u}}{\partial t} = c_v \frac{\partial^2 \bar{u}}{\partial z^2} \quad \dots(15.17)$$

where  $c_v = \text{coefficient of consolidation} = \frac{k}{m_v \gamma_w}$  ...(15.18)

$$= \frac{k(1 + e_0)}{a_v \gamma_w} \quad \dots(15.19)$$

Eq. 15.17 is the basic differential equation of consolidation which relates the rate of change of excess hydrostatic pressure to the rate of expulsion of excess pore water from a unit volume of soil during the same time interval. The term *coefficient of consolidation*  $c_v$  used in the equation is adopted to indicate the combined effects of permeability and compressibility of soil on the rate of volume change. The units of  $c_v$  are  $\text{cm}^2/\text{sec}$ .

## 15.6. SOLUTION OF THE CONSOLIDATION EQUATION\*

The solution of the differential equation of consolidation is obtained by means of the Fourier series. The solution must satisfy the following hydraulic boundary conditions [Fig. 15.5 (a)] :



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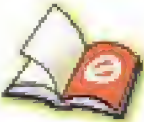


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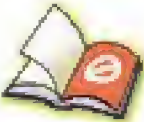
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$$c_v = \frac{(T_v)_{90} d^2}{t_{90}}$$

where  $t_{90} = 30.1$  minutes  
 $= 30.1 \times 60$  seconds

$(T_v)_{90}$  = times factor  
 corresponding to 90%  $U$   
 $= 0.848$  (Table 15.1)

$d$  = average drainage path  
 for the pressure increment

$$= \frac{H}{2} = \frac{1}{2} \left[ \frac{H_1 + H_f}{2} \right]$$

$$= \frac{1.738 + 1.611}{4} = 0.837 \text{ cm}$$

$$c_v = \frac{0.848 (0.837)^2}{30.1 \times 60}$$

$$= 3.94 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

**2. Logarithm of time fitting method.** This method suggested by A. Casagrande (1930), is based on the characteristic of the  $U - \log_{10} T_v$  plot in which intersection of a tangent at the point of inflection and the asymptote of the lower portion is at the ordinate of 100%  $U$ . This characteristic is used to determine the point of 100%  $U$  on the semi-log plot of laboratory time-consolidation curve. Fig. 15.11 shows the theoretical curve while Fig. 15.12 shows the plotting of the test data of Table 15.5.

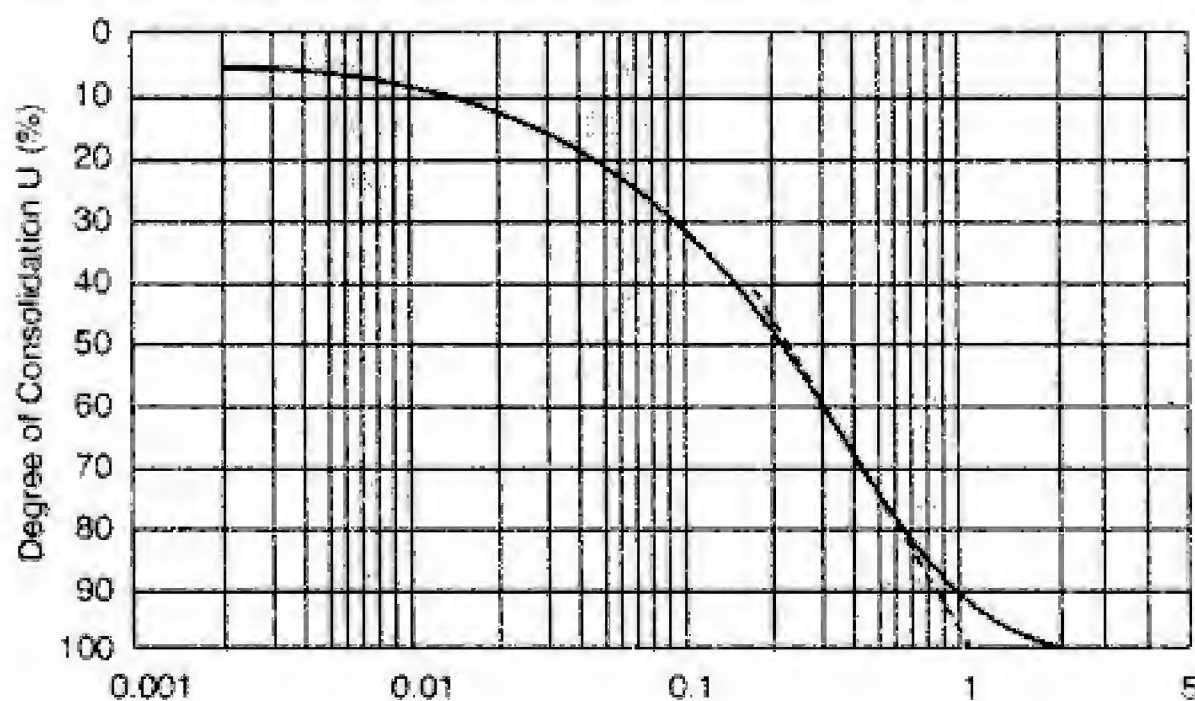


FIG. 15.11. THEORETICAL CURVE  $\log T_v$  AND  $U$  (REF. TABLE 15.1).

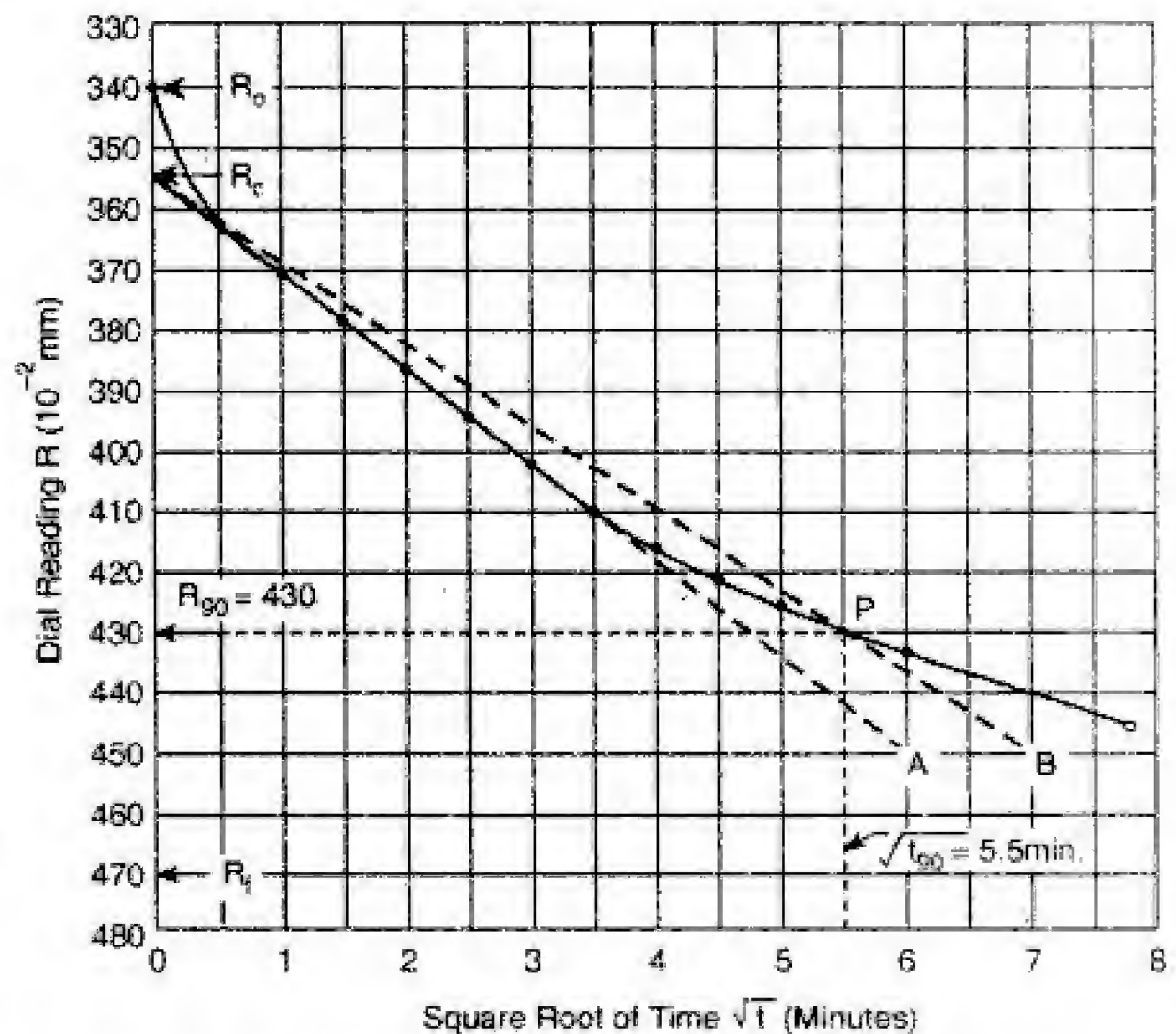
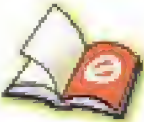


FIG. 15.10. TIME-CONSOLIDATION CURVE (TABLE 15.5).





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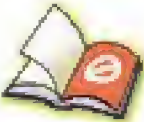
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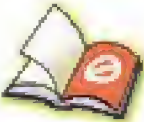
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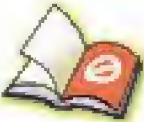




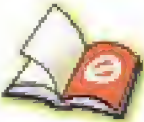
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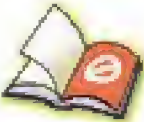
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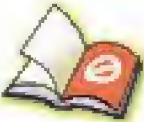




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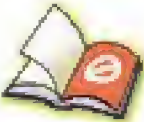
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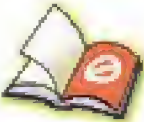
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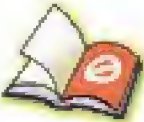
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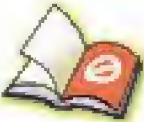




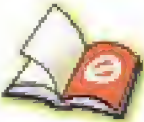
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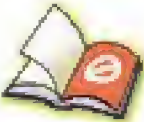
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2. **Voids ratio.** The voids ratios at the end of various pressure are calculated from Eq. 15.37 :  $e = \frac{H - H_s}{H_s}$ . Table 15.3. illustrates the method of calculating voids ratios at various applied pressure.

Results are tabulated as shown in Table 15.7.

3. **Coefficient of consolidation.** The coefficient of consolidation at each pressure increment is calculated by the following equation :

$$c_v = \frac{0.197 d^2}{t_{50}} \text{ (Log fitting methods)} \quad \text{and} \quad c_v = \frac{0.848 d^2}{t_{90}} \text{ (Square root fitting method)}$$

In the log fitting method, a plot is made between dial reading and logarithm of time (Fig. 15.12) and the corresponding to 50% consolidation is determined. In the square root fitting method, a plot is made between dial reading and square to root time (Fig. 15.10) and the time corresponding to 90% consolidation is determined. The values of  $c_v$  are recorded in Table 15.7.

4. **Compression index.** To determine the compression index, a plot of voids ratio  $e$  versus  $\log \sigma'$  is made (Fig. 15.3). The initial compression curve would be found to be a straight line, and the slope of this line would give the compression index  $C_c$ .

5. **Coefficient of compressibility.** The coefficient of compressibility is calculated as follows :

$$a_v = \frac{0.435 C_c}{\sigma'} \quad \text{where } a_v = \text{coefficient of compressibility ; } \sigma' = \text{average pressure for the increment}$$

6. **Coefficient of permeability.** The coefficient of permeability  $k$  is calculated as follows:

$$k = \frac{c_v a_v \gamma_w}{1 + e}$$

**Note 1. Specimen size.** IS : 2720 (part XV)-1986 recommends specimen diameter of 60 mm, though specimens of diameters 50, 70 and 100 mm may also be used in special cases. The specimen should be at least 20 mm thick in all cases. However, the thickness should not be less than 10 times the maximum diameter of the grain in the soil specimen. The diameter of thickness ratio should be a minimum of 3.

2. **Sample preparation.** While preparing the specimen, attempt should be made to have the soil strata oriented in the same direction in the consolidation apparatus as they were or will be oriented in practice. The orientation should also be such that the laboratory test normally compresses the soil in the same direction relative to soil strata as the applied load in the field. In trimming the soil, great care should be taken to handle the specimen delicately with the least pressure applied to the soil.

## PROBLEMS

1. The foundation of a new structure is to be laid on a bed of sand overlying a layer of saturated clay 1.5 metres thick. Below the clay layer is another bed of coarse sand. The effective overburden pressure at the middle of the clay layer before construction of the foundation is  $100 \text{ kN/m}^2$ . Due to foundation loading an additional increase of  $100 \text{ kN/m}^2$  in vertical pressure is expected at the middle of the clay layer.

An undisturbed sample of clay, 2 cm thick was tested in a floating ring consolidation apparatus with opportunity of drainage both ways. Under a pressure of  $100 \text{ kN/m}^2$ , the total change in thickness was 0.06 cm and under  $200 \text{ kN/m}^2$ , it was 0.1 cm. The time taken for 50% of the total settlement to occur under the  $200 \text{ kN/m}^2$  pressure was 1 hour. Compute (1) Coefficient of volume decrease for the range of 100 to  $200 \text{ kN/m}^2$ , (2) ultimate settlement of clay bed, (3) time required for 50% of the ultimate settlement.

[Ans (1)  $0.206 \text{ m}^2/\text{MN}$  ; (2) 3.09 cm ; (3) 234 days]

2. During a consolidation test, a sample of fully saturated clay, 2.0 cm thick is consolidated under a pressure increment of  $200 \text{ kN/m}^2$ . When equilibrium is reached, the sample thickness gets reduced to 1.78 cm. The



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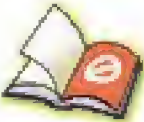


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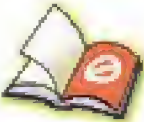
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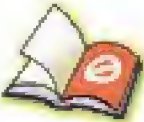
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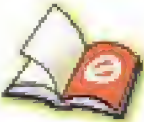


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achieved. For proper compaction control as the work progresses, rapid methods of testing must be used.

The rapid determination of water content can be done by two methods: (i) the calcium carbide method discussed in § 3.2 and (ii) the Proctor needle method. The second method is commonly used in the field. The Proctor needle consists of a needle point, attached to graduated needle shank which in turn is attached to a spring loaded plunger. The needle point of varying cross-sectional area are available so that a wide range of penetration resistance can be measured. The penetration force is read on a loaded gauge fixed over the handle [Fig. 17.6 (a)].

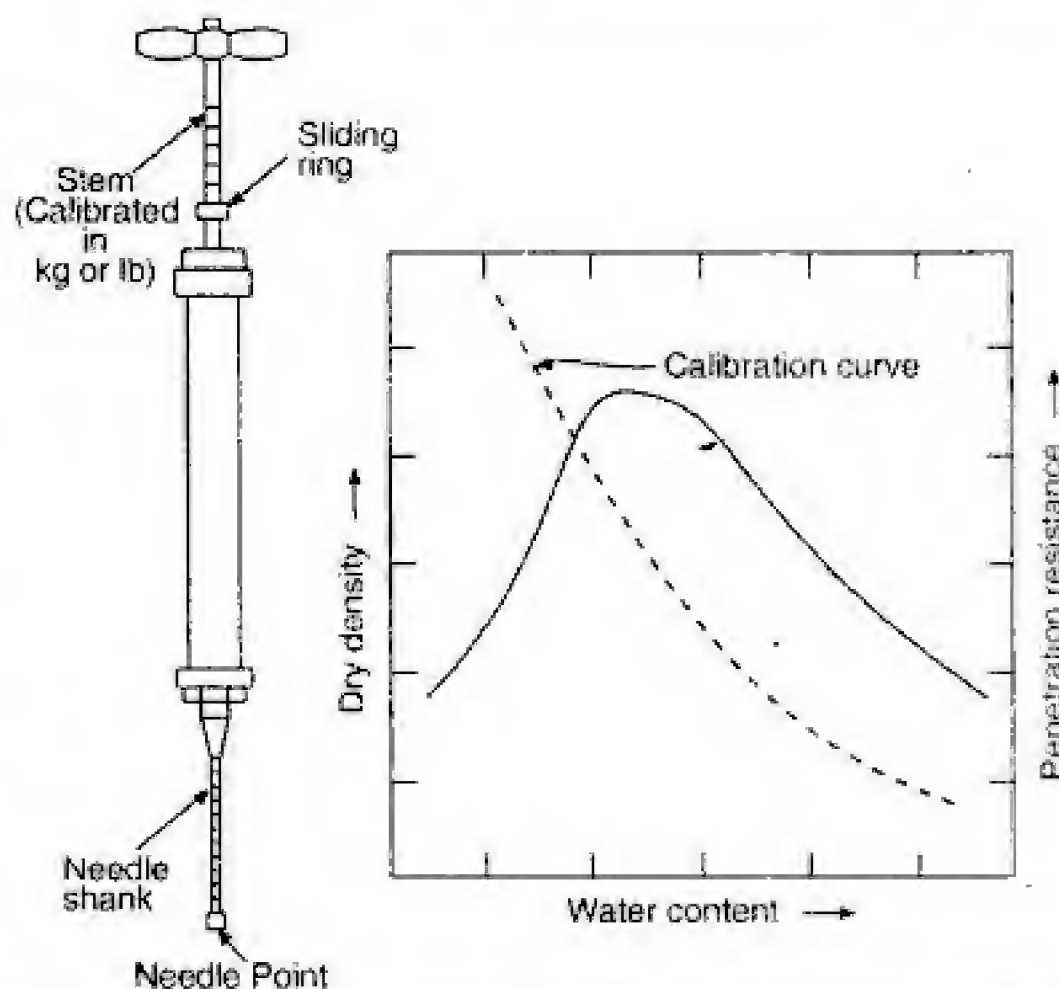


FIG. 17.6. PROCTOR NEEDLE AND CALIBRATION CURVE.

To use the needle in the field, a calibration curve [Fig. 17.6 (b)] is plotted in the laboratory between the penetration resistance as the ordinate and the water content as the abscissa. The laboratory penetration resistance is measured by inserting the Proctor needle in the compacted soil in the Proctor mould. The penetration resistances corresponding to various water contents are thus noted at the end of each Proctor compaction, and a calibration curve is plotted. This curve may be used to determine the placement water content. The penetration resistance of the compacted soil in the field is determined with the Proctor's needle, and its water content is read off from the calibration curve.

The bulk density of the compacted soil in the field can be determined by any of the methods described in chapter 3. The degree of compaction obtained in the field is measured by the *relative compaction* or the *per cent compaction* which is defined as the ratio, expressed as a percentage, of the field dry density to the maximum density obtained in a laboratory compaction test. In free-draining cohesionless soils, the degree of compaction is sometimes expressed in terms of density index, particularly in earth dam construction.

### 17.11. FACTORS AFFECTING COMPACTION

The various factors which affect the compacted density are as follows : (i) water content, (ii) amount and type of compaction, (iii) method of compaction (iv) type of soil, and (v) addition of admixtures.

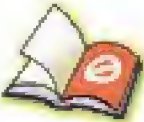
**1. Water content :** It has been seen by laboratory experiments that as the *water content* is increased, the compacted density goes on increasing, till a maximum dry density is achieved after which further addition of water decreases the density. When only a



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empty mass is 944 g. If the water content of the embankment soil is 8% and the other test data are as given below, determine the in-situ dry density :

- (i) mass of excavated soil = 925 g
- (ii) mass of cylinder + sand before the test = 5332 g
- (iii) mass of cylinder + sand after filling the hole = 4152 g
- (iv) mass of sand filling the pouring cone = 432 g
- (v) mass of calibrating can plus sand = 2483 g.

[Ans.  $1.76 \text{ g/cm}^3$ ]





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The co-ordinates of points on the circle represent the normal and shearing stresses on inclined planes at a given point. This circle is known as *Mohr's circle of stress* (Mohr, 1870).

To draw the Mohr circle, the normal stresses  $\sigma_x$  and  $\sigma_y$  are marked on the abscissa, at points  $B$  and  $A$  and a circle is drawn with point  $C$ , mid-way between  $A$  and  $B$ , as the centre, with radius equal to  $CB_1 = CA_1$  where  $BB_1$  and  $AA_1$  are the perpendiculars drawn at  $B$  and  $A$  of magnitude equal to  $\tau_{xy}$ . The sign conventions are shown in Fig. 18.1(b). Fig 18.1 (c) shows the Mohr circle so drawn. The co-ordinates of any point  $F(\sigma, \tau)$  represent the stress conditions on plane which makes an angle  $\alpha$  with the  $x$  direction.

If from a point  $B_1$  [Fig. 18.1 (d)] on a circle representing the state of stress on vertical plane, a line is drawn parallel to this plane (*i.e.* vertical), it intersects the circle at a point  $P$ . Also, if from the point  $A_1$  on the circle representing the stresses on the horizontal plane, a line is drawn parallel to this latter plane (*i.e.* horizontal) it will also intersect the circle in the same point  $P$ . In general, if through a point  $F$  representing the stresses on a given plane, a line is drawn parallel to that plane, it will also intersect the circle in the point  $P$ . The point  $P$  is therefore, a unique point called the *origin of planes* or the *pole*.

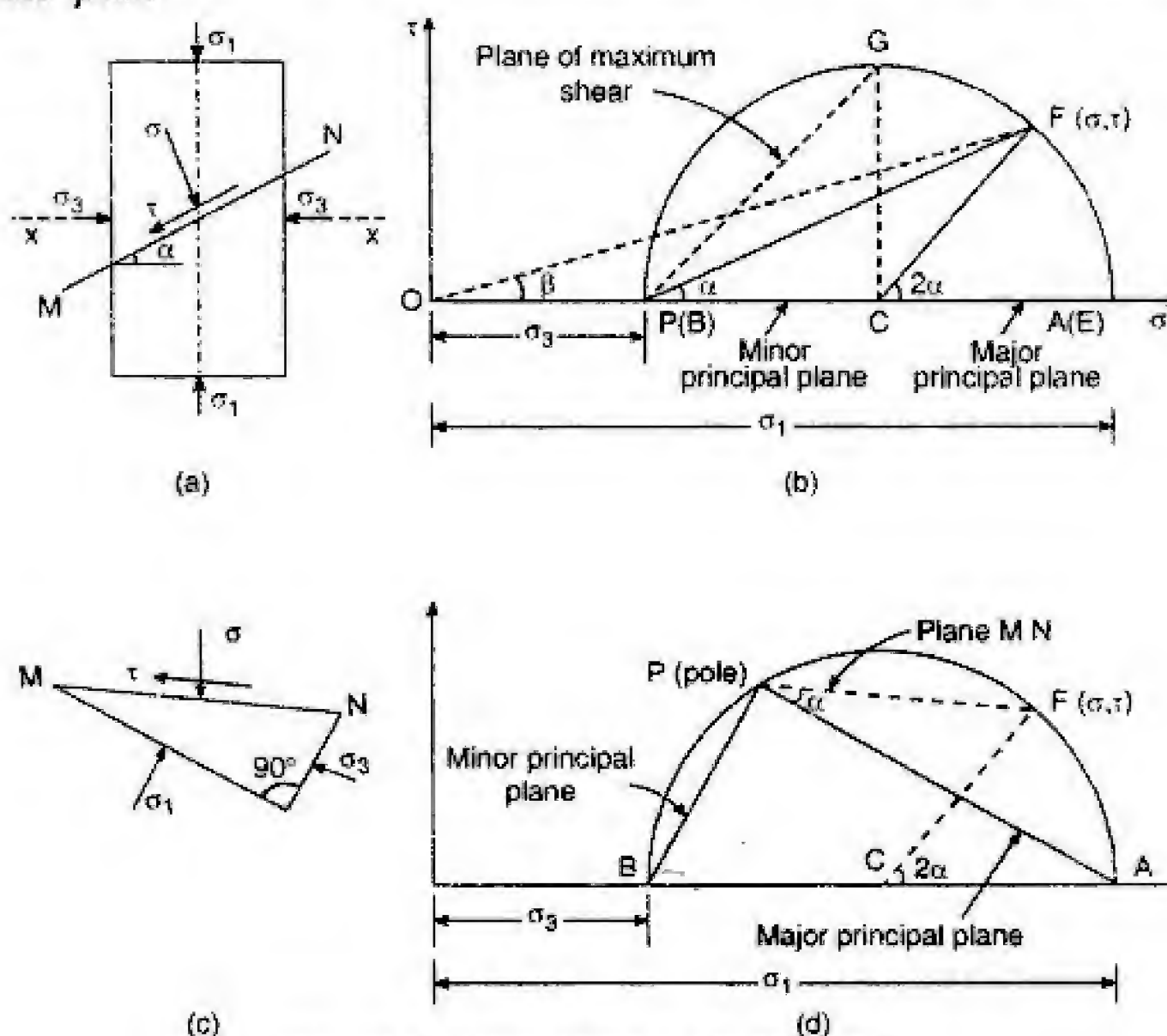


FIG. 18.2 MOHR CIRCLE OF STRESSES.

Let us now take the case of soil element whose sides are the principal planes, *i.e.* consider the state of stress where only normal stresses are acting on the faces of the element. Fig. 18.2 (a) shows the element, and Fig. 18.2 (b) shows the Mohr circle.



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line  $\tau_f = c' + \sigma' \tan \phi'$ . The pole  $P$  will be the point with stress co-ordinates as  $(\sigma_3', 0)$ . The Mohr circle is tangential to the Mohr envelope at the point  $F$ .  $PF$  represents the direction of the failure plane, inclined at an angle  $\alpha_f$  with the direction of the major principal plane. From the geometry of Fig. 18.5, we get from triangle  $JFK$

$$2\alpha_f = 90^\circ + \phi' \quad \text{or} \quad \alpha_f = 45^\circ + \frac{\phi'}{2}$$

It should be noted that for any combination of the applied principle effective stress  $\sigma_1'$  and  $\sigma_3'$ , failure will occur only if the stress circle touches the failure envelope. Also, the co-ordinates of the failure point  $F$  represent the stress components  $\sigma'$  and  $\tau_f$  at failure. As it is evident from Fig. 18.5, the  $\tau_f$  at failure is less than the maximum shear stress, corresponding to the point  $G$ , acting on the

plane  $PG$ . Thus, the failure plane does not carry maximum shear stress, and the plane which has the maximum shear stress is *not* the failure plane.

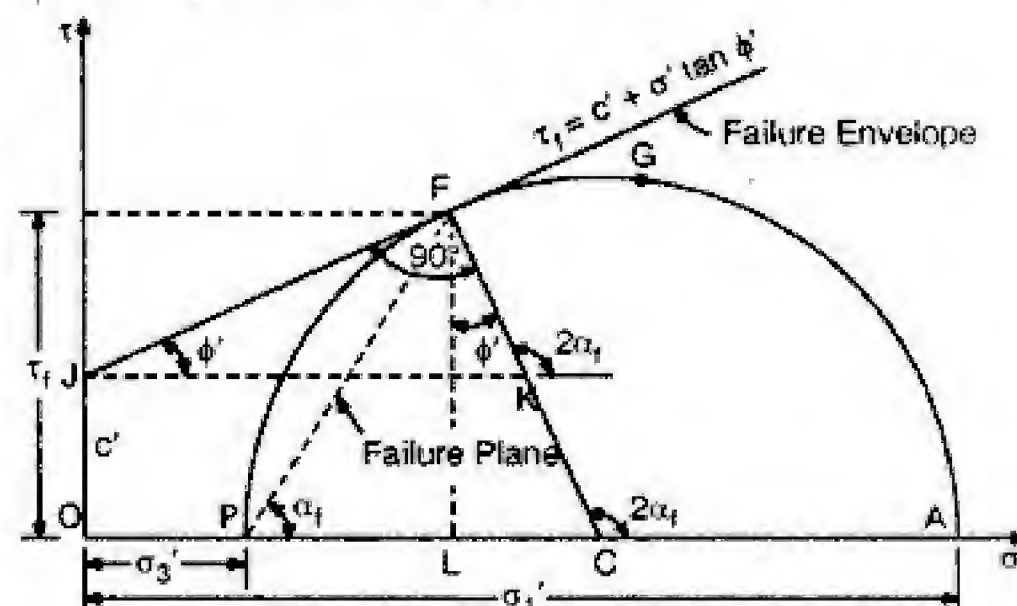


FIG. 18.5

### 18.5. MEASUREMENT OF SHEAR STRENGTH

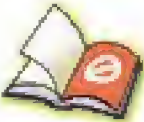
The measurement of shear strength of soil involves certain test observations at failure with the help of which the *failure envelope* or *strength envelope* can be plotted corresponding to a given set of conditions (specially the drainage conditions). Shearing resistance can be determined in the laboratory by the following four methods :

- |                                 |                         |
|---------------------------------|-------------------------|
| (1) Direct shear test           | (2) Triaxial shear test |
| (3) Unconfined compression test | (4) Vane shear test.    |

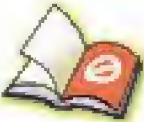
Again depending upon the *drainage conditions*, three types of shear tests have been developed :

- (a) Undrained test or quick test  
 (b) Consolidated undrained test  
 and (c) Drained test.

In the *undrained* or *quick test*, no drainage of water is permitted. Hence there is no dissipation of pore pressure during the entire test. In the *drained test*, drainage is permitted throughout the test during the application of both normal and shear stresses, so that full consolidation occurs and no excess pore pressure is set up at any stage of the test. In the consolidated-undrained test, drainage is permitted under the initially applied normal stress only and full primary consolidation or softening is allowed to take place. No drainage is allowed afterwards. The parameters  $c$  and  $\phi$  are not fundamental properties of the soil; they may simply be considered merely coefficients derived from the geometry of the graph obtained by the plotting shear stress at failure against normal stress. They vary with drainage conditions of the test.



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outlet from the top of the specimen. A separate compressor is used to apply fluid pressure in the cell. Pore pressure developed in the specimen during the test can be measured with the help of a separate pore pressure measuring equipment, such as Bishop's apparatus shown in Fig. 18.8. The cylindrical specimen is enclosed in a rubber membrane. A stainless steel piston running through the centre of the top cap applies the vertical compressive load (called the *deviator stress*) on the specimen under test. The load is applied through a proving ring, with the help of a mechanically operated load frame. Depending upon the drainage conditions of the test, solid nonporous discs or end caps, or porous discs are placed on the top and bottom of the specimen and the rubber membrane is sealed on to these end caps by rubber rings.

The length of the specimen is kept about 2 to  $2\frac{1}{2}$  times its diameter. The cell pressure  $\sigma_3 (= \sigma_2)$  acts all round the specimen; it acts also on the top of the specimen as well as the vertical piston meant for applying the deviator stress. The vertical stress applied by the loading frame, through the proving ring is equal to  $(\sigma_1 - \sigma_3)$ , so that the total stress on the top of the specimen  $= (\sigma_1 - \sigma_3) + \sigma_3 = \sigma_1 =$  major principal stress. This principal stress difference  $(\sigma_1 - \sigma_3)$  is called the *deviator stress* recorded on the proving ring dial. Another dial measures the vertical deformation of the sample during testing. It is desirable to maintain the cell pressure reservoir and mercury control apparatus, devised by Skempton and Bishop (1950), as shown in Fig. 18.9 (a). For long duration test (lasting about a week or more), self-compensating mercury control can be used [Fig. 18.9 (c)].

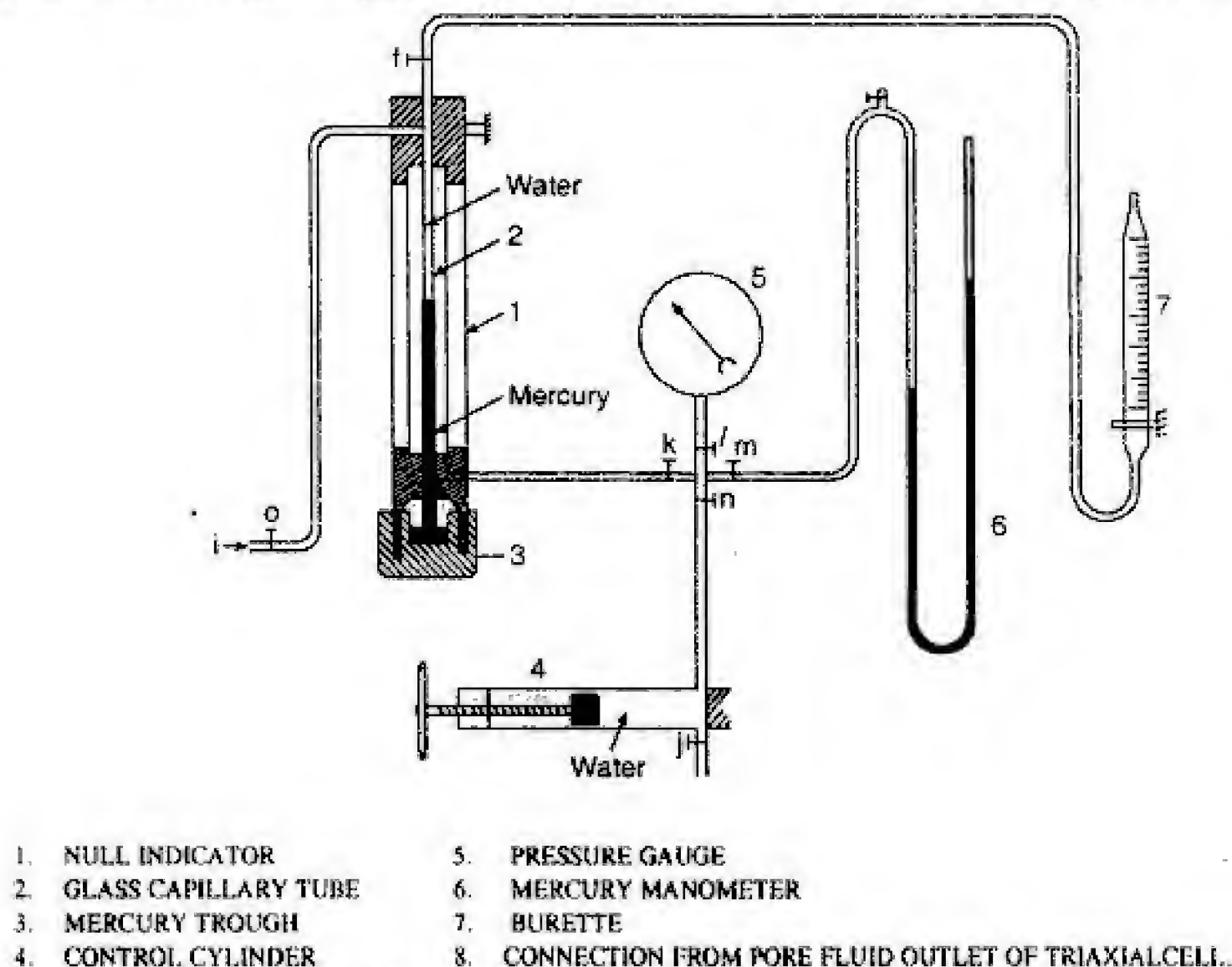


FIG. 18.8. LAYOUT OF BISHOP'S PORE PRESSURE APPARATUS (BISHOP 1950, 1961)



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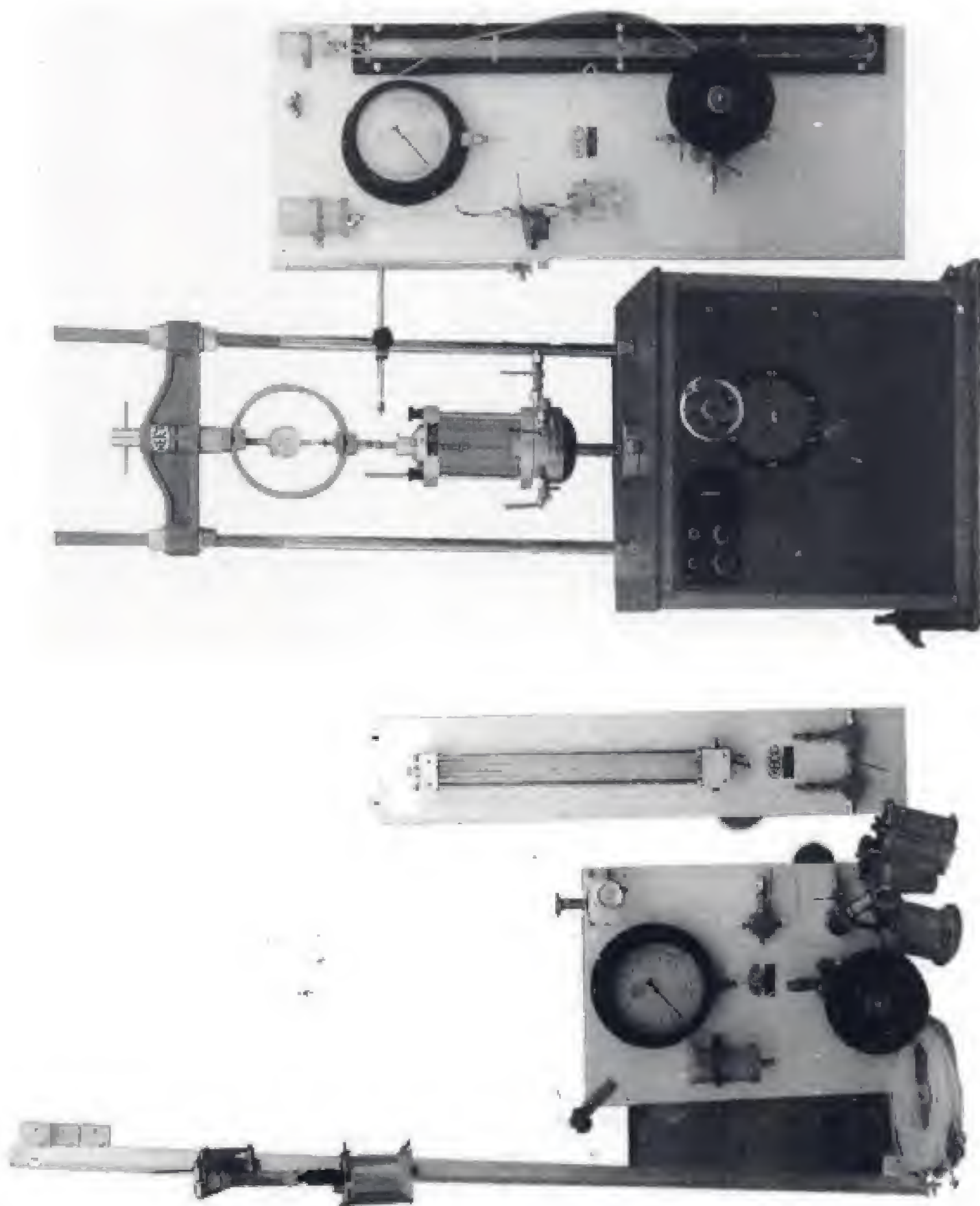
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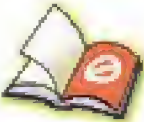


TRIAXIAL SHEAR TEST OUTFITS (HEICO)

PLATE VIII



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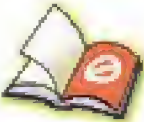
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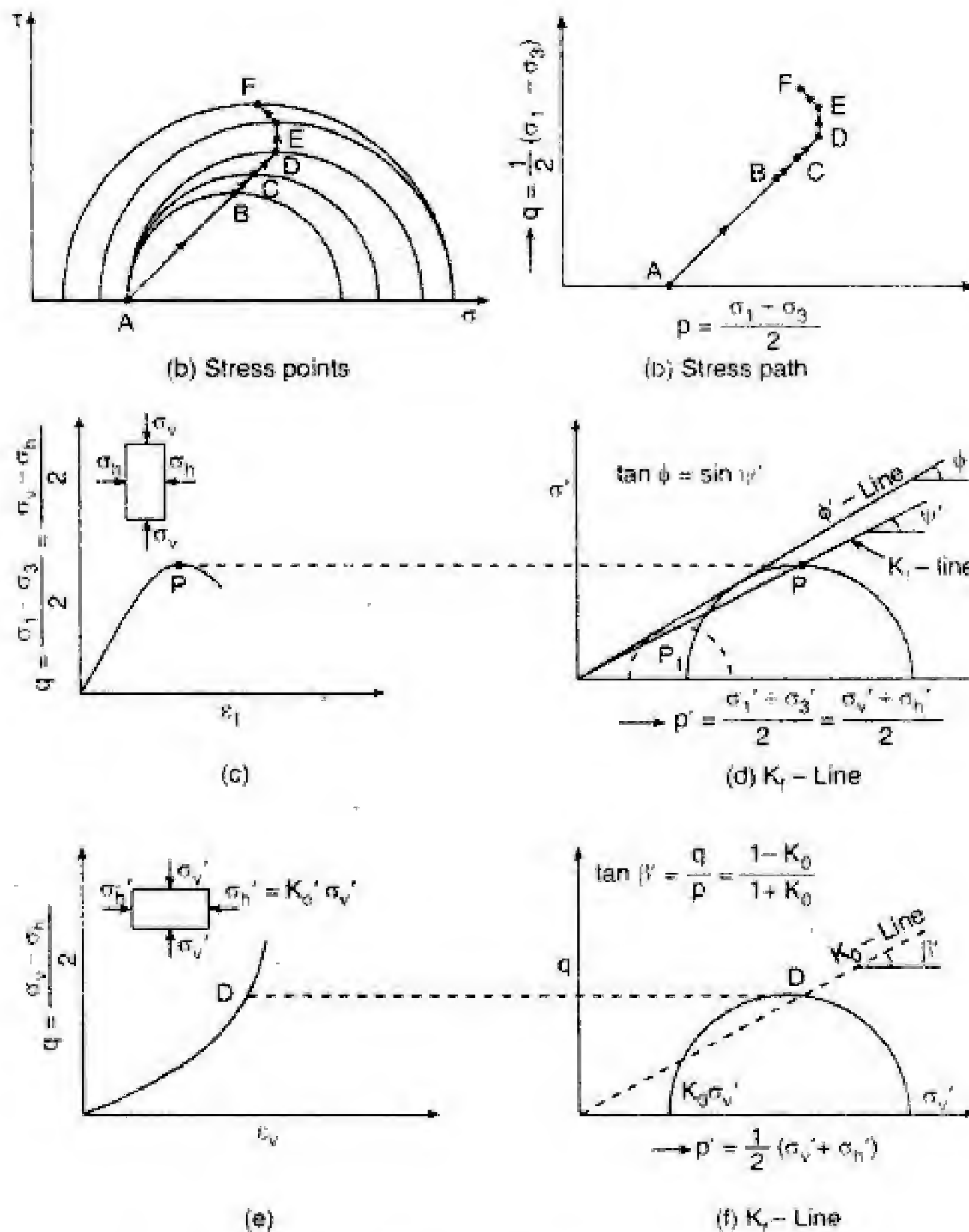
FIG. 18.36. LAMBE'S STRESS PATH :  $K_f$  AND  $K_0$  LINES.

Fig. 18.36 (c) and (d) correspond to three dimensional compression in the laboratory. The line joining the origin to the point P of maximum shear stress corresponding to the Mohr circle at failure, represents the  $K_f$  - line. It is a line joining all such points of maximum shear of Mohr circles at failures to the origin. Its inclination  $\psi'$  to the horizontal bears a fixed relation to the inclination  $\phi'$  of the failure envelope (called the  $\phi'$  line)

$$\sin \psi' = \tan \phi' \quad \dots(18.45)$$

Fig. 18.36 (e) and (f) correspond to the case of confined compression (oedometer) with no horizontal movement. The corresponding stress path  $K_0$  indicates the way in which



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**Solution:** Given :  $\sigma_3 = 1.2 \text{ kg/cm}^2$  ;  $\sigma_d = 4.0 \text{ kg/cm}^2$   
 $\therefore \sigma_1 = \sigma_3 + \sigma_d = 1.2 + 4 = 5.2 \text{ kg/cm}^2$   
 $\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha$

Since  $c = 0$ ,  $\sigma_1 = \sigma_3 \tan^2 \alpha$

$\therefore \tan \alpha = \sqrt{\sigma_1 / \sigma_3} = \sqrt{5.2 / 1.2} = 2.082$

$\therefore \alpha = 64.34^\circ = 45^\circ + \phi/2$  or  $\phi = 2(64.34^\circ - 45^\circ) \approx 38.7^\circ$

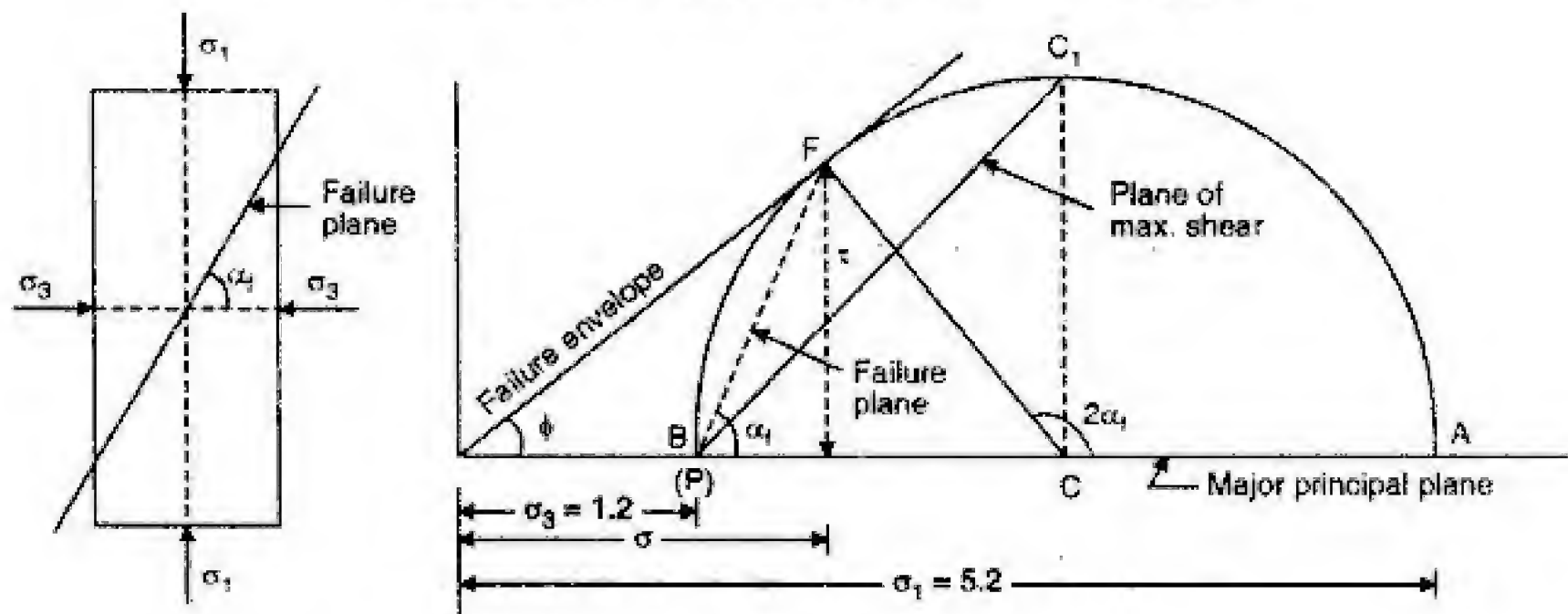


FIG. 18.46

(i) Hence angle of shearing resistance  $= 38.7^\circ$

(ii) Shear stress  $\tau = FF_1 = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha = \frac{4}{2} \sin (2 \times 64.34^\circ) = 1.56 \text{ kg/cm}^2$

Normal stress  $\sigma = OF_1 = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha$   
 $= \frac{5.2 + 1.2}{2} + \frac{5.2 - 1.2}{2} \cos (2 \times 64.34^\circ) = 1.95 \text{ kg/cm}^2$

(iii) Angle made by the failure plane with the major principal plane  $\alpha_f = 64.34^\circ$

$\therefore$  Angle made by the failure plane with the minor principal plane  $= 90 - \alpha$   
 $= 90 - 64.34^\circ = 25.66^\circ$

(iv) Maximum shear stress,  $\tau = \frac{\sigma_1 - \sigma_3}{2} = \frac{5.2 - 1.2}{2} = 2 \text{ kg/cm}^2$

Angle made by the plane of maximum shear stress with the major principal plane

$$= \angle C_1 B C = \tan^{-1} \frac{CC_1}{BC} = \tan^{-1} \frac{2}{2} = 45^\circ$$

**Example 18.33.** A shear vane of 7.5 cm diameter and 11.0 cm length was used to measure the shear strength of soft clay. If torque of 600 kg-cm was required to shear the soil, calculate the shear strength. The vane was then rotated rapidly to cause remoulding of the the soil. The torque required in the remoulded state was 200 kg-cm. Determine the sensitivity of the soil and offer your comments on the same. (Engg. Services Exam, 1992)

**Solution :** Refer Example 18.11.

$$\tau_f = \frac{T}{\pi d^2 \left[ \frac{H}{2} + \frac{d}{6} \right]} = \frac{600}{\pi (7.5)^2 \left[ \frac{11}{2} + \frac{7.5}{6} \right]} = 0.503 \text{ kg/cm}^2$$



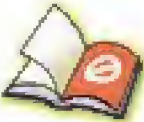
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## Failure Envelopes\*

### 19.1. INTRODUCTION

For the case of plain strain, the condition of limiting equilibrium can be expressed by the following equations:

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} = 0 \quad \dots(19.1 \ a)$$

$$\frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \sigma_z}{\partial z} = \gamma \quad \dots(19.1 \ b)$$

where  $\sigma_x$ ,  $\sigma_z$  are the normal stress, and  $\tau_{xz}$  is the shearing stress on the plane. From Eq. 19.1, it is clear that conditions of equilibrium alone are not sufficient to provide the stress field within the soil body since there are three unknown stresses. In order to relate these three stresses in a unique way, another independent expression should exist, so that the resulting system could fulfil the needs for assessing the adequacy of a structure subjected to loading. Since such an expression has to provide a measure of the circumstances governing motion of a part of the soil body, it is said to be a *yield or failure criterion*. The yield criterion may be expressed as

$$f(I_1, I_2, I_3) = 0 \quad \dots(19.2)$$

where  $I_1, I_2, I_3$  are the invariants of stress. For the assumptions of homogeneity and isotropy, it can also be expressed as

$$F(\sigma_1, \sigma_2, \sigma_3) = 0$$

where  $\sigma_1$ ,  $\sigma_2$  and  $\sigma_3$  are the principal stresses given by the roots of Segner's equation

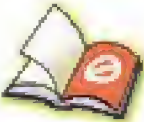
$$\sigma^3 - I_1 \sigma^2 + I_2 \sigma - I_3 = 0$$

If all the stress combinations sufficient to cause failure are represented by the above fundamental relationship, they could trace out a surface called the *yield or failure surface or failure envelope*. In as much as any combination of stress falling on the surface represents a limiting condition or state of failure, the determination of its shape is of paramount importance.

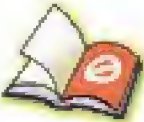
The shape of the failure surface, in a three dimensional system of co-ordinates ( $\sigma_1, \sigma_2, \sigma_3$ ) that Eq. 19.3 represents, can be seen from Fig. 19.1.  $OA$  represents the space diagonal or the hydrostatic axis along which  $\sigma_1 = \sigma_2 = \sigma_3$  inclined at  $\alpha = \cos^{-1} 1/\sqrt{3} = 54^\circ 44'$  with the three axes. Experiments conducted by Crossland (1965) have demonstrated that a pure state of



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It is clear that the ratio is independent of  $c$  and approaches unity with decreasing value of  $\phi$  as shown in Table 19.1.

TABLE 19.1 RATIO OF  $O'A$  AND  $O'A'$  FOR MOHR-COULOMB FAILURE SURFACE

$\phi$	$0^\circ$	$5^\circ$	$10^\circ$	$15^\circ$	$20^\circ$	$25^\circ$	$30^\circ$	$35^\circ$	$40^\circ$
$\frac{O'A}{O'A'}$	1.000	0.943	0.889	0.842	0.796	0.753	0.715	0.678	0.647
$\frac{O'A'}{O'A}$	1.000	1.061	1.123	1.188	1.258	1.328	1.400	1.473	1.546

For  $\phi = 0$  the Mohr-Coulomb criterion reduces to  $\sigma_1 - \sigma_3 = c$  ... (19.8 a)

or  $\sigma_{max} - \sigma_{min} = \text{constant} = 2k$  (say) ... (19.8 b)

or  $\tau_{max} = \frac{\sigma_{max} - \sigma_{min}}{2} = k$  ... (19.8 c)

This criterion was first given by Tresca in 1968. He assumes that failure occurs for given material when the maximum shear stress equals to a critical constant value. The corresponding failure surface is a prism, for which the right section is a regular hexagon with  $O'A = O'A'$ . The extended Tresca condition, credited to Sandels, is expressed by

$$[(\sigma_1 - \sigma_2)^2 - \{c + k_1 \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)\}^2] [(\sigma_2 - \sigma_3)^2 - \{c + k_1 \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)\}^2] \\ \times [(\sigma_3 - \sigma_1)^2 - \{c + k_1 \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)\}^2] = 0 \quad \dots (19.8d)$$

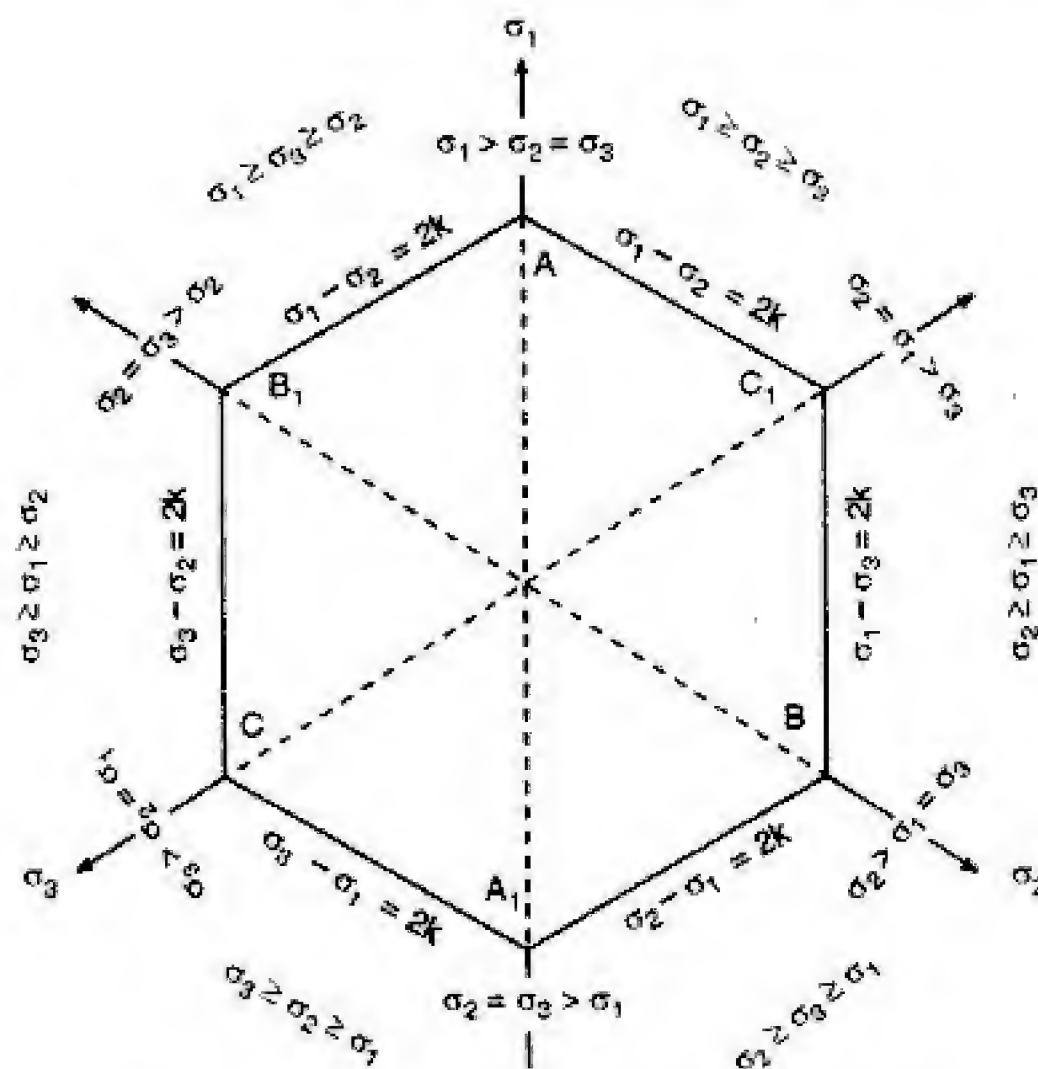


FIG. 19.6. THE TRESCA FAILURE ENVELOPE.





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# Earth Pressure

## 20.1. INTRODUCTION

In the design of retaining walls, sheet piles, or other earth-retaining structures, it is necessary to compute the lateral pressure exerted by the retained mass of soil. The question of finding out the lateral earth pressure against retaining walls is one of the oldest in the civil engineering field. The *plastic state of stress* when the failure is imminent was investigated by Rankine in 1860. A lot of theoretical and experimental work has been done in this field, and many theories and hypotheses have been proposed.

A *retaining wall* or retaining structure is used for maintaining the ground surfaces at different elevations on either side of it. The material retained or supported by the structure is called *backfill* which may have its top surface horizontal or inclined. The position of the backfill lying above a horizontal plane at the elevation of the top of a wall is called the *surcharge*, and its inclination to the horizontal is called *surcharge angle*  $\beta$ .

## 20.2 PLASTIC EQUILIBRIUM IN SOILS : ACTIVE AND PASSIVE STATES

A body of soil is said to be in plastic equilibrium if every point of it is on the verge of failure. Rankine investigated the stress conditions corresponding to those states of plastic equilibrium which can be developed simultaneously throughout a semi-infinite mass of soil acted on by no force other than gravity. The stress condition during plastic equilibrium can be represented by the following Mohr-Coulomb equation :

$$\frac{\sigma_1 - \sigma_3}{2} - \frac{\sigma_1 + \sigma_3}{2} \sin \phi = c \cos \phi \quad \dots(20.1)$$

$$\text{or} \quad \sigma_1 = 2c \tan(45^\circ + \phi/2) + \sigma_3 \tan^2(45^\circ + \phi/2) \quad \dots(20.2)$$

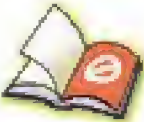
$$\text{or} \quad \sigma_1 = 2 c \sqrt{N_\phi} + \sigma_3 N_\phi \quad \dots(20.3)$$

where  $N_\phi = \tan^2(45^\circ + \phi/2) =$  flow value

$\sigma_1$  and  $\sigma_3$  = major and minor principal stresses at any point in the soil mass, at failure.

In terms of stress components in  $x$ - $z$  plane, Eq. 20.1 may be written as :

$$\sqrt{\left(\frac{\sigma_z - \sigma_x}{2}\right)^2 + \tau_{xz}^2} - \frac{\sigma_z + \sigma_x}{2} \sin \phi = c \cos \phi \quad \dots(20.4)$$



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For loose sands, the first two conclusions remain valid, but the horizontal compression required to reach full passive resistance may be as large as 15% .

**Effective stress analysis.** If the analysis is based on effective stresses, Eqs. 20.8 and 20.9 are modified as under :

$$K_a = \frac{\sigma_h'}{\sigma_v'} = \frac{1}{\tan^2 (45^\circ + \phi'/2)} = \frac{1 - \sin \phi'}{1 + \sin \phi'} \quad \dots(20.8 \ a)$$

$$K_p = \frac{\sigma_h'}{\sigma_v'} = \frac{1 + \sin \phi'}{1 - \sin \phi'} \quad \dots(20.9 \ a)$$

### 20.3. EARTH PRESSURE AT REST\*

The earth pressure at rest, exerted on the back of a rigid, unyielding retaining structure, can be calculated using theory of elasticity, assuming the soil to be semi-infinite, homogeneous, elastic and isotropic. Consider an element of soil at a depth  $z$ , being acted upon by vertical stress  $\sigma_v$  and horizontal stress  $\sigma_h$ . There will be no shear stress. The lateral strain  $\epsilon_h$  in the horizontal direction is given by :

$$\epsilon_h = \frac{1}{E} [ \sigma_h - \mu (\sigma_h + \sigma_v) ]$$

The earth pressure at rest corresponding to the condition of zero lateral strain ( $\epsilon_h = 0$ ).

$$\text{Hence} \quad \sigma_h = \mu (\sigma_h + \sigma_v) \quad \text{or} \quad \frac{\sigma_h}{\sigma_v} = K_0 = \frac{\mu}{1 - \mu} \quad \dots(20.10)$$

where  $K_0$  is *coefficient of the earth pressure at rest*.

Designating the lateral earth pressure ( $\sigma_h$ ) at rest by  $p_0$  and substituting  $\sigma_v = \gamma z$ , we have

$$p_0 = K_0 \gamma z$$

The pressure distribution diagram is thus triangular with zero intensity at  $z = 0$  and an intensity of  $K_0 \gamma H$  at the base of the wall, where  $z = H$ . The total pressure  $P_0$  per unit length for the vertical height  $H$  is given by

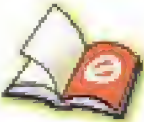
$$P_0 = \int_0^H K_0 \gamma z \cdot dz = \frac{1}{2} K_0 \gamma H^2 \quad \dots(20.12)$$

The behaviour of soil is not in accordance with the elastic theory and do not have a well-defined value of the Poisson's ratio. Table 20.1 gives some value of  $K_0$  available from experience.

TABLE 20.1. VALUES OF  $K_0$  FOR DIFFERENT SOILS

Soil Type	$K_0$
1. Loose sand	0.4
2. Dense sand	0.6
3. Sand compacted in layers	0.8
4. soft clay	0.6
5. Hard clay	0.5





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$$\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \sin \phi \quad \dots(1)$$

Fig. 20.8 (b) shows that Mohr circle corresponding to the principal stress intensities  $\sigma_1$  and  $\sigma_3$  at  $A$ . The Mohr's circle is the locus of a point that represents that resultant stress intensity at all planes passing through the point under consideration. Hence we can find the resultant stress intensities  $\sigma$  and  $p$  (i.e. conjugate stresses) on the two conjugate planes at point  $A$ . The obliquity of  $\sigma$  and  $p$  is  $\beta$ . Hence through origin  $O$ , draw a line at obliquity  $\beta$  to cut the circle at  $A_1$  and  $A_2$ . Then  $OA_1$  represents the resultant stress  $p$  and  $OA_2$  represents the resultant stress  $\sigma$ . Draw  $OB$  perpendicular to  $A_1 A_2$ .

$$\begin{aligned} OB &= OC \cos \beta \\ &= \frac{\sigma_1 + \sigma_3}{2} \cos \beta \dots(2) \end{aligned}$$

$$BC = OC \sin \beta = \frac{\sigma_1 + \sigma_3}{2} \sin \beta \quad \dots(3)$$

$$A_1 B = B A_2 = \sqrt{A_1 C^2 - BC^2} = \sqrt{\left(\frac{\sigma_1 - \sigma_3}{2}\right)^2 - \left(\frac{\sigma_1 + \sigma_3}{2}\right)^2 \sin^2 \beta}$$

But  $(\sigma_1 - \sigma_3) = (\sigma_1 + \sigma_3) \sin \phi$ , from (1) ; Hence

$$A_1 B = B A_2 = \frac{\sigma_1 + \sigma_3}{2} \sqrt{\sin^2 \phi - \sin^2 \beta} \quad \dots(4)$$

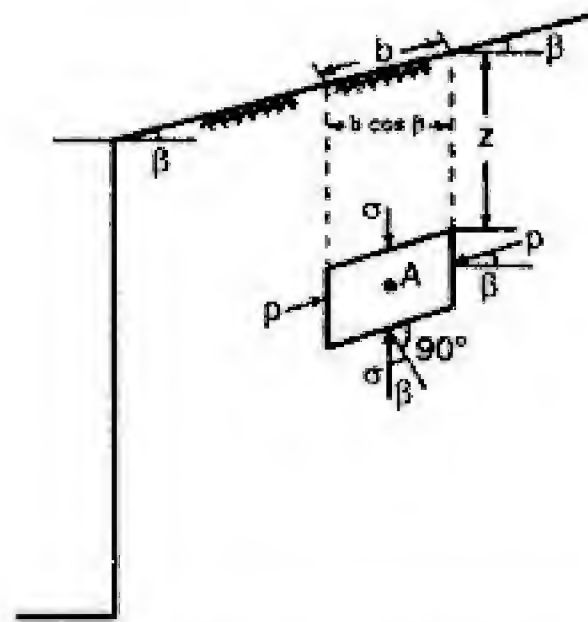
$$\text{Now stress } \sigma = OB + B A_2 = \frac{\sigma_1 + \sigma_3}{2} \cos \beta + \frac{\sigma_1 + \sigma_3}{2} \sqrt{\sin^2 \phi - \sin^2 \beta} \quad \dots(5)$$

$$\text{and stress } p = OB - A_1 B = \frac{\sigma_1 + \sigma_3}{2} \cos \beta - \frac{\sigma_1 + \sigma_3}{2} \sqrt{\sin^2 \phi - \sin^2 \beta} \quad \dots(6)$$

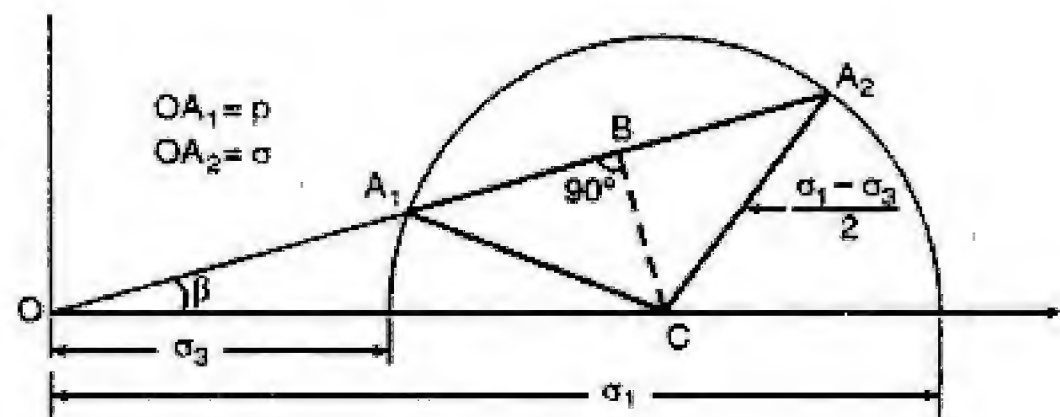
Dividing (5) and (6), we get

$$\frac{p}{\sigma} = K = \frac{\cos \beta - \sqrt{\sin^2 \phi - \sin^2 \beta}}{\cos \beta + \sqrt{\sin^2 \phi - \sin^2 \beta}} = K = \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \quad \dots(20.23)$$

The ratio  $K$  is called the *conjugate ratio* or Rankine's lateral pressure ratio.

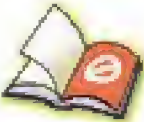


(a) Pressures on the soil element



(b) Mohr-circle

FIG. 20.8 CONJUGATE STRESS RELATIONSHIP.



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$$z = 3 \text{ m}, \quad p_a = 54 - 48 = 6 \text{ kN/m}^2 \text{ (lower soil)}$$

$$z = 6 \text{ m}, \quad p_a = 54 + 20(6 - 3) - 48 = 66 \text{ kN/m}^2$$

The pressure distribution is shown in Fig. 20.15 (a).

Assuming that the tension cracks have been developed the *net* active pressure will be equal to the area of the positive pressure diagram.

$$P_1 = \frac{1}{2} \times 1 \times 18 = 9 \text{ kN/m}$$

Acting at  $3 + \frac{1}{3} = 3.33 \text{ m}$  from the base

$$P_2 = \frac{6 + 66}{2} \times 3 = 108 \text{ kN/m}$$

Acting at  $\frac{3}{3} \left( \frac{2 \times 6 + 66}{6 + 66} \right) = 1.083 \text{ m}$  from the base.

Total pressure  $P = P_1 + P_2$   
 $= 9 + 108 = 117 \text{ kN/m}$  length.

Distance of the point of application of  $P$  from base is given by

$$\bar{z} = \frac{(9 \times 3.33) + (108 \times 1.083)}{117} = 1.26 \text{ m.}$$

**Example 20.7.** A retaining wall, 4 m high supports a backfill ( $c = 20 \text{ kN/m}^2$ ;  $\phi = 30^\circ$ ;  $\gamma = 20 \text{ kN/m}^3$ ) with horizontal top, flush with the top of the wall. The backfill carries a surcharge of  $20 \text{ kN/m}^2$ . If the wall is pushed towards the backfill, compute the total passive pressure on the wall, and its point of application.

**Solution.**  $c = 2 \text{ kN/m}^2$

$$\phi = 30^\circ; K_p = N_\phi = \tan^2 (45^\circ + \phi/2) = \tan^2 60^\circ = 3$$

The passive pressure intensity due to surcharge is given by

$$p_1 = K_p \cdot q = N_\phi q = 3 \times q = 3 \times 20 = 60 \text{ kN/m}^2$$

The passive pressure intensity due to backfill is given by Eq. 20.46 a :

$$p = 2c \sqrt{N_\phi} + \gamma H N_\phi = p_2 + p_3$$

where

$$p_2 = 2c \sqrt{N_\phi} = 2 \times 20 \sqrt{3} = 69.28 \text{ kN/m}^2$$

and

$$p_3 = \gamma H N_\phi = 20 \times 4 \times 3 = 240 \text{ kN/m}^2$$

The pressure distribution diagram consists of two rectangles and one triangle corresponding to  $p_1$ ,  $p_2$  and  $p_3$ , as shown in Fig. 20.16.

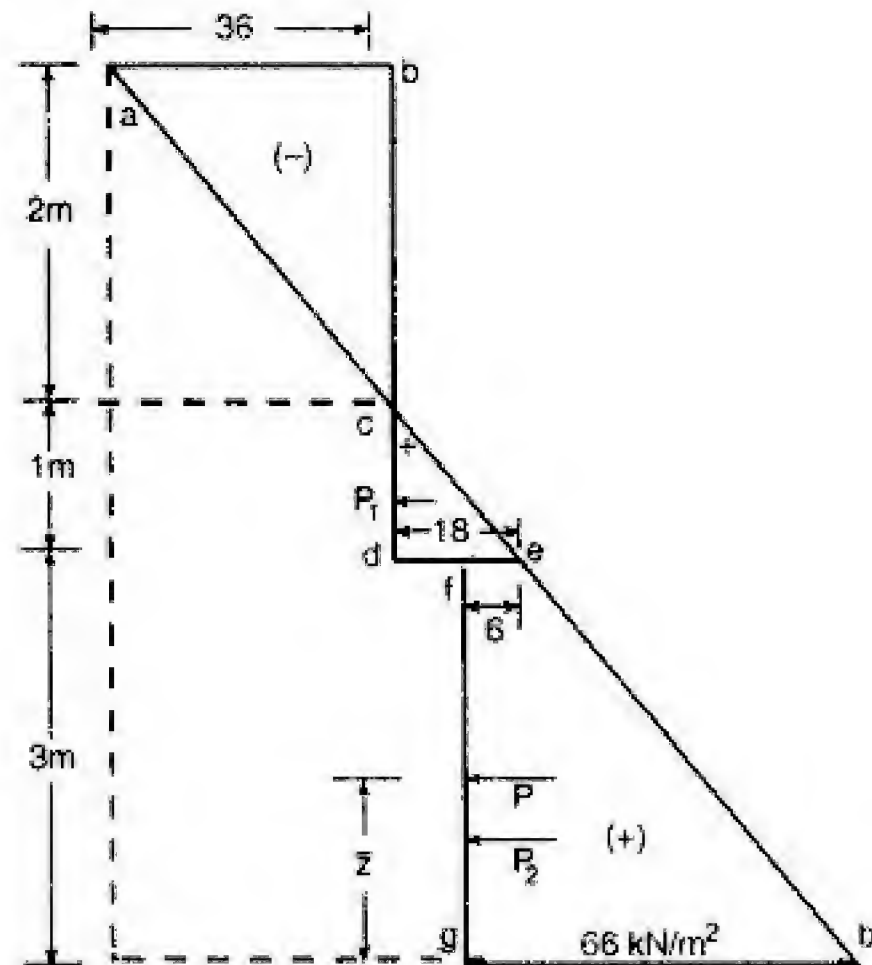


FIG. 20.15 (a)





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or the *critical* slip surface is that for which the wall reaction is maximum, *i.e.*, the wall must resist the maximum lateral pressure before it moves away from the fill.

**Condition for maximum pressure from a sliding wedge.**  $BD$  shows a plane inclined at an angle  $\phi$  to the horizontal at which the soil is expected to stay in the absence of any lateral support. The line  $BD$ , therefore, is called the natural slope line, *repose line*

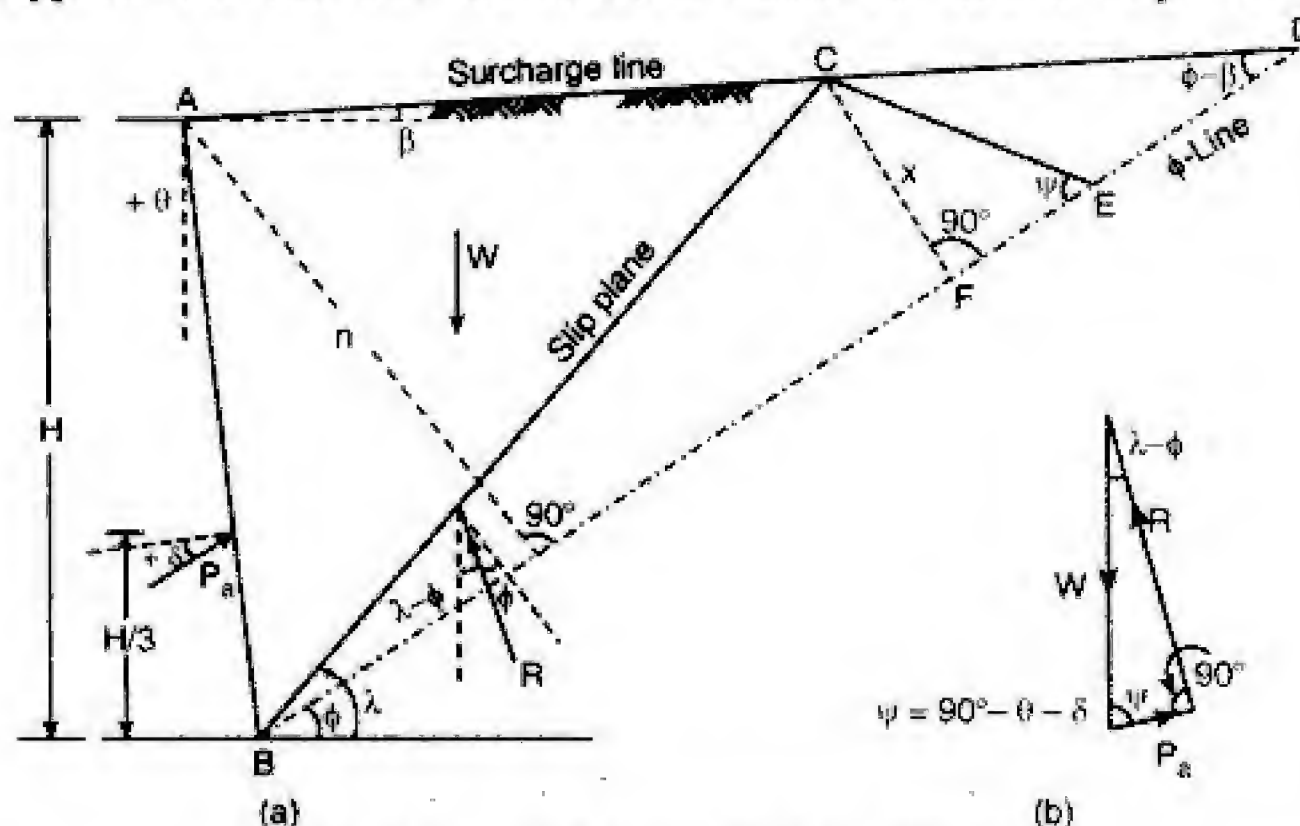


FIG. 20.20. COULOMB'S WEDGE THEORY.

or the  $\phi$  - line.  $AD$ , inclined at  $\beta$  to the horizontal, is called the *ground line* or *surcharge line*. Plane  $BC$ , inclined at angle  $\lambda$  (to be determined) is the  $S$  line or rupture plane or *slip plane*; the angle  $\lambda$  is called the *critical slip angle*. The reaction  $R$  inclined at an angle  $\phi$  to the normal to the slip line;  $R$  is also inclined at an angle  $(\lambda - \phi)$  to the vertical. The wall reaction  $P_a$  is inclined at an angle  $\delta$  to the normal to the wall. The inclination of  $P_a$  to vertical is represented by angle  $\psi = 90^\circ - \theta - \delta$  (= constant for given value of  $\theta$  and  $\delta$ ). The force triangle is shown in Fig 20.20 (b). The value of  $P_a$  depends upon the slip angle  $\lambda$ .  $P_a$  is zero when  $\lambda = \phi$ . As  $\lambda$  increases beyond  $\phi$ ,  $P$  also increases and after reaching a maximum value it again reduces to zero when  $\lambda$  equals  $90^\circ + \theta$ . Thus, the critical slip plane lies between  $\phi$ -line and the back of the wall.

In order to derive the condition for maximum active pressure  $P_a$  from the sliding wedge, draw line  $CE$  at an angle  $\psi$  to the  $\phi$ -line. Let  $x$  and  $n$  be the perpendicular distances of points  $C$  and  $A$  from the  $\phi$ -line, and  $m$  be the length of line  $BD$ . It will be seen that triangle  $BCE$  and the force triangle [Fig. 20.20 (b)] are similar.

$$\text{Hence} \quad \frac{P_a}{W} = \frac{CE}{BE} \quad \therefore \quad P_a = W \cdot \frac{CE}{BE} \quad (I) \dots (20.50)$$

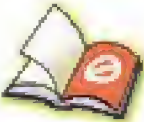
$$\text{Now,} \quad CE = x \operatorname{cosec} \psi = A_1 x, \quad (\text{where } A_1 = \operatorname{cosec} \psi = \text{constant})$$

$$BE = BD - (DF - FE) = m - x [\cot (\phi - \beta) - \cot \psi] = m - A_2 x$$

$$\text{where} \quad A_2 = \cot (\phi - \beta) - \cot \psi = \text{constant}$$



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$$\therefore p_a = \frac{2 P_a}{AB} = \frac{2 P_a \cdot \cos \theta}{H} = \frac{2 \times 36 \times \cos 8^\circ}{3} = 23.8 \text{ kN/m}^2$$

Fig. 20.26 shows the pressure distribution diagram.

**Example 20.12.** Solve example 20.11 taking  $\beta = 20^\circ$ .

**Solution :** Fig. 20.22 shows the Rebhann's construction with the given data.

By measurement  $CE = 2.13 \text{ m}$  and  $CF = x = 2.06 \text{ m}$

$$\therefore P_a = \frac{1}{2} \times \gamma (CE \cdot x) = \frac{1}{2} \times 19 (2.13 \times 2.06) = 41.6 \text{ kN/m length.}$$

**Example 20.13.** Solve example 20.11 taking  $\beta = \phi = 20^\circ$ .

**Solution :** Fig. 20.23 shows the Rebhann's construction with the given data. By measurement, we get

$$CE = 3 \text{ m} \quad CF = x = 2.9 \text{ m.}$$

$$\therefore P_a = \frac{1}{2} \times 19 \times 3 \times 2.9 = 83 \text{ kN/m.}$$

**Example 20.14.** A masonry retaining wall 6 m high is back filled with granular soil having angle of internal friction of  $32^\circ$ . The back face (i.e. soil face) of the wall is inclined to the vertical at a positive batter angle of  $12^\circ$  and the back fill is sloping upward from the top of the wall at a slope of 10:1. Assuming the angle of wall friction as  $16^\circ$ , calculate the total active earth pressure on the wall per metre length. The backfill has water content of 16%, degree of saturation of 70% and specific gravity of 2.68.

**Solution**

$$\tan \beta = \frac{1}{10}, \text{ which gives } \beta = 5.71^\circ$$

$$e = \frac{wG}{S} = \frac{0.16 \times 2.68}{0.7} = 0.613$$

$$\begin{aligned} \therefore \gamma_d &= \frac{G \gamma_w}{1 + e} = \frac{2.68 \times 9.81}{1 + 0.613} \\ &= 16.3 \text{ kN/m}^3 \end{aligned}$$

$$\begin{aligned} \text{and } \gamma &= \gamma_d (1 + w) = 16.3 (1 + 0.16) \\ &= 18.91 \text{ kN/m}^3 \end{aligned}$$

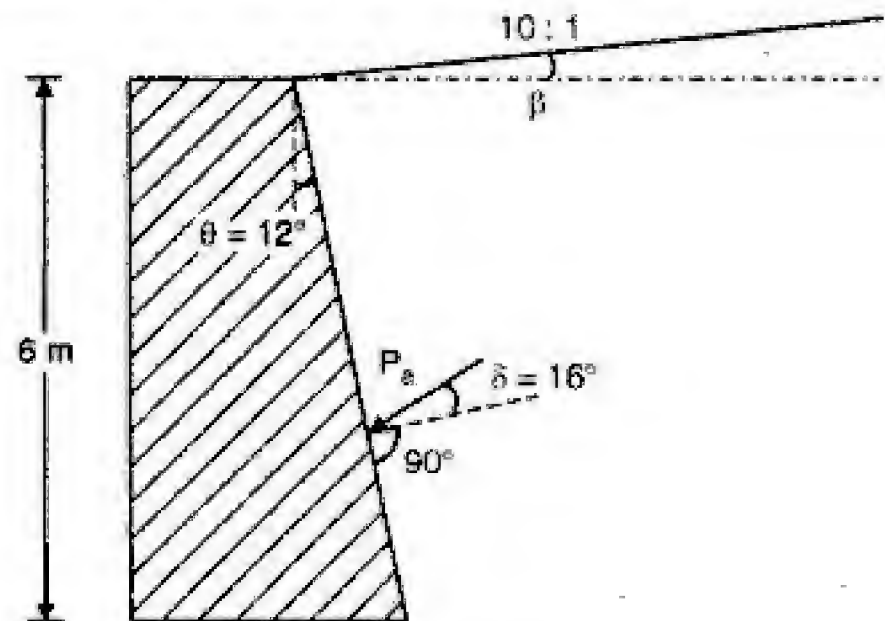


FIG. 20.27

$$K_a = \left[ \frac{\sec \theta \cos (\phi - \theta)}{\sqrt{\cos (\theta + \delta)} + \sqrt{\frac{\sin (\phi + \delta) \sin (\phi - \beta)}{\cos (\beta - \theta)}}} \right]$$

$$\theta = 12^\circ, \quad \sec \theta = 1.0223; \quad \phi - \theta = 32 - 12^\circ = 20^\circ; \quad \cos (\phi - \theta) = 0.9397$$

$$\theta + \delta = 12 + 16 = 28^\circ; \quad \cos (\theta + \delta) = 0.8829; \quad \phi + \delta = 32^\circ + 16^\circ = 48^\circ; \quad \sin (\phi + \delta) = 0.7431$$

$$\phi - \beta = 32^\circ - 5.71^\circ = 26.29^\circ; \quad \sin (\phi - \beta) = 0.4429; \quad \beta - \theta = 5.71^\circ - 12^\circ; \quad \cos (\beta - \theta) = 0.994$$





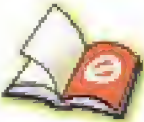
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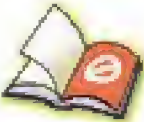
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$3b_1$ , where  $b_1$  is the distance of the point of application of  $R$  from end  $C$ . The intensity  $f_1$  will then be given by

$$f_1 = \frac{2 R_v}{3 b_1} \quad \dots(20.67 \text{ a})$$

Substituting  $b_1 = \frac{b}{2} - e$ ,  $f_2 = \frac{2 R_v}{3 (b - 2e)} \quad \dots(20.67 \text{ b})$

The following are the criteria of design of gravity retaining wall:

1. The maximum pressure  $f_1$  should not exceed the bearing capacity of soil.
2. For no tension to develop,  $e < b/6$ , or  $\bar{x} \leq \frac{2b}{3} \quad \dots(20.68)$

3. For no sliding to occur,  $R_h < R_v \cdot \mu \quad \dots(20.69)$

where  $\mu = \tan \delta' =$  coefficient of friction between the base of the wall and the soil.

The factor of safety  $F$  against sliding is given by

$$F = \frac{R_v \cdot \mu}{R_h} \quad \dots(20.70)$$

The minimum value of  $F$  should be 1.5.

4. For the wall to be stable against overturning,  $R$  must pass within the base width. However, if the requirement of no tension is fulfilled, complete safety against overturning is automatically assured.

**Example 20.19.** Design a gravity retaining wall, 5 m high with vertical back to retain a dry cohesionless backfill of unit weight  $18 \text{ kN/m}^3$  and angle of shearing resistance  $30^\circ$ . Find also the factor of safety against sliding assuming the angle of friction between the base of the wall and the foundation soil as  $30^\circ$ . The wall is to be 1 m wide at top, and to be constructed of brick masonry having unit weight  $20 \text{ kN/m}^3$ . Use Rankine's theory.

**Solution.**

$$\phi = 30^\circ \therefore K_a = \frac{1}{3}$$

$$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2} \times \frac{1}{3} \times 18 (5)^2 = 75 \text{ kN/m}$$

It acts horizontally at a height of  $\frac{5}{3}$  m from the base.

Let  $b_1 =$  width of wall at its top

$b =$  width of wall at its bottom

Since  $P_a$  acts horizontally, we have

$$R_v = W \text{ and } R_h = P_a$$

For no tension to develop,  $R_v$  must act at a distance  $2b/3$  from heel  $B$ .

From statics, the moment of the forces  $W$  and  $P_a$  about  $B$  must be equal to the moment of the resultant about  $B$ . For calculating  $W$ , it is convenient to divide the

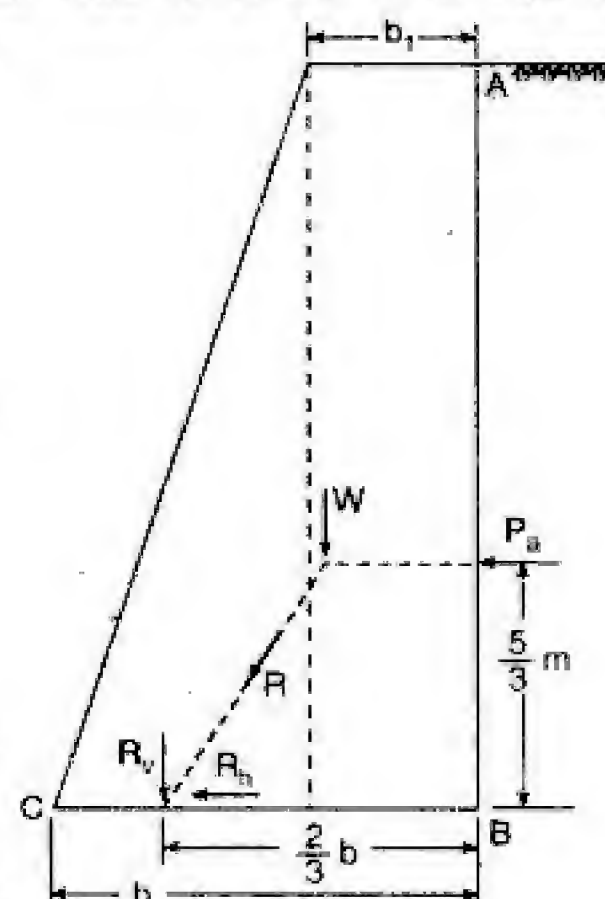


FIG. 20.36.



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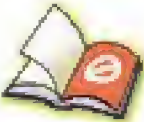
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**Example 20.34.** Determine the stresses at the top and bottom of a vertical cut, 4.5 m deep in soil with  $\phi' = 16^\circ$  and  $c' = 1.91 \text{ t/m}^2$  ( $19.1 \text{ kN/m}^2$ ) and  $\gamma = 1.85 \text{ t/m}^3$  ( $18.5 \text{ kN/m}^3$ ). What could be the depth of the potential crack? What is the maximum depth of excavation that can be left unsupported. (Engg. Services Exam. 1990)

**Solution :** For a  $c - \phi$  soil, the stress (i.e. active earth pressure) at any depth  $z$  is given

by 
$$p_a = \gamma z \cot^2 \alpha' - 2 c' \cot \alpha' = \gamma z K_a - 2 c' \sqrt{K_a}$$

Here, 
$$K_a = \frac{1 - \sin 16^\circ}{1 + \sin 16^\circ} = 0.5679$$

$$\therefore p_a = 18.5 z (0.5679) - 2 \times 19.1 \sqrt{0.5679} = 10.505 z - 28.787 \quad \dots(1)$$

At top,  $z = 0$ . Hence  $p_a = -28.787 \text{ kN/m}^2$  (i.e. tension)

At bottom,  $z = 4.5 \text{ m}$ ,  $p_a = 10.505 \times 4.5 - 28.784 = 18.489 \text{ kN/m}^2$

Depth of potential crack is at  $z_0$  where  $p_a = 0$

$$\therefore z_0 = \frac{2 c'}{\gamma \sqrt{K_a}} = \frac{2 \times 19.1}{18.5 \sqrt{0.5679}} = 2.74 \text{ m}$$

Max. depth of unsupported excavation  $= 2 z_0 = 2 \times 2.74 = 5.48 \text{ m}$

**Example 20.35.** A smooth vertical wall 4 m high is pushed against a mass of soil having a horizontal surface and a shearing resistance given by Coulomb's equation in which  $c = 20 \text{ kN/m}^2$  and  $\phi = 30^\circ$ . The unit weight of the soil is  $20 \text{ kN/m}^3$ . Its surface carries a uniform load of  $20 \text{ kN/m}^2$ . What is the total passive Rankine's pressure? What is the distance from the base of the wall to the centre of the pressure? (Engg. Services Exam. 1994)

**Solution :** This problem is exactly the same as Example 20.6. It was therefore set from Authors present book. For solution, see Example 20.6

**Example 20.36.** A vertical bank was formed during the excavation of a soil having  $\phi = 15^\circ$  and unit weight of  $1800 \text{ kg/m}^3$ . When the depth of excavation reached 5.5 m the bank failed. What was the approximate value of cohesion of clay? (Engg. Services Exam. 1994)

**Solution :** 
$$z_0 = \frac{2c}{\gamma \sqrt{K_a}} \quad \text{where } K_a = \frac{1 - \sin 15^\circ}{1 + \sin 15^\circ} = 0.589$$

$$z_0 = \frac{2c}{1800 \sqrt{0.589}} = 1.448 \times 10^{-3} c$$

$\therefore$  Safe depth of cutoff  $= 2 z_0 = 2.896 \times 10^{-3} c$

$\therefore 2.896 \times 10^{-3} c = 5.5$  From which  $c = 1899 \text{ kg/m}^2$

**Example 20.37.** A counterfort wall of 10 m height retains non-cohesive backfill. The void ratio and angle of internal friction of the backfill respectively are 0.7 and  $30^\circ$  in the loose state and they are 0.40 and  $40^\circ$  in the dense state. Calculate and compare active and passive earth pressure in both the states. Take specific gravity of soil grains as 2.7. (Engg. Services Exam. 1995)

**Solution :** (a) Loose state :  $e = 0.70$ ;  $\phi = 30^\circ$  ;  $G = 2.7$  ;

$$K_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3} ; K_p = 3 ; \quad \gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.7 \times 9.81}{1 + 0.7} = 15.58 \text{ kN/m}^3$$

$$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2} \times \frac{1}{3} \times 15.58 (10)^2 = 259.67 \text{ kN/m}$$

$$P_p = \frac{1}{2} K_p \gamma H^2 = \frac{1}{2} \times 3 \times 15.58 (10)^2 = 2337 \text{ kg/m}$$

(b) Dense state :  $e = 0.40$  and  $\phi = 40^\circ$ ;  $G = 2.7$  as before.



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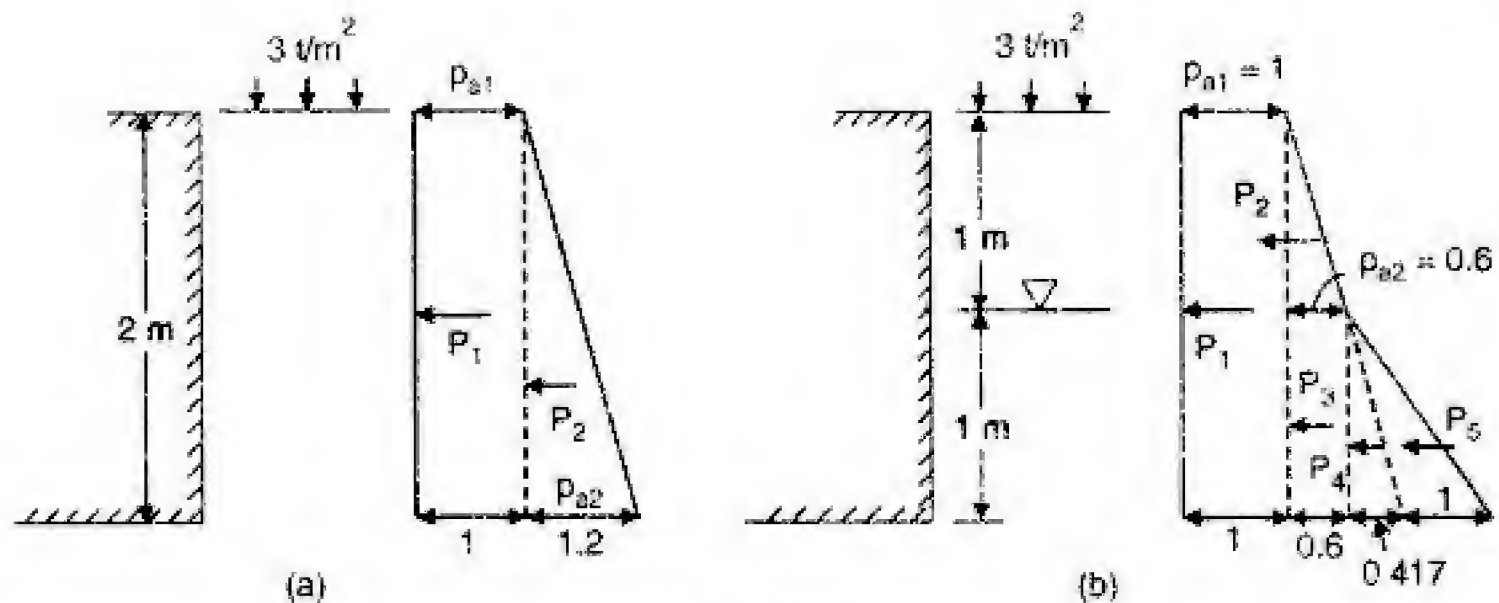


FIG. 20.53

Acting at  $z = \frac{2 \times 1 + 1.2 \times 0.667}{3.2} = 0.875$  m above base.

(ii) Water table at 1 m above base of wall [Fig. 20.53 (b)]

$$p_{a1} = K_a q = \frac{1}{3} \times 3 = 1 \text{ t/m}^2; \quad \therefore p_{a2} = K_a \gamma H_1 = \frac{1}{3} \times 1.8 \times 1 = 0.6 \text{ t/m}^2$$

$$p_{a3} = p_{a2} = 0.6 \text{ t/m}^2; \quad p_{a4} = K_a \gamma' H_2 = \frac{1}{3} \times 1.25 \times 1 = 0.417 \text{ t/m}^2; \quad p_{a5} = \gamma_w H_2 = 1(1) = 1 \text{ t/m}^2$$

$$\therefore P_1 = 1 \times 2 = 2 \text{ t/m acting at } 2/2 = 1 \text{ m above base}$$

$$P_2 = \frac{1}{2} \times 0.6 \times 1 = 0.3 \text{ t/m acting at } 1 + 1/3 = 1.333 \text{ m above base}$$

$$P_3 = 0.6 \times 1 = 0.6 \text{ t/m acting at } 1/2 = 0.5 \text{ m above base}$$

$$P_4 = \frac{1}{2} \times 0.417 \times 1 = 0.209 \text{ t/m acting at } 1/3 = 0.333 \text{ m above base}$$

$$P_5 = \frac{1}{2} \times 1 \times 1 = 0.5 \text{ t/m acting at } 1/3 = 0.333 \text{ m above base}$$

$$\text{Total } P_a = 2 + 0.3 + 0.6 + 0.209 + 0.5 = 3.609 \text{ t/m}$$

$$\text{Acting at } z = \frac{2 \times 1 + 0.3 \times 1.333 + 0.6 \times 0.5 + 0.209 \times 0.333 + 0.5 \times 0.333}{3.609} = 0.814 \text{ m above base}$$

**Example 20.42.** A smooth retaining wall 6 m high retains dry granular backfill weighing  $16 \text{ kN/m}^3$  to its level surface. The active thrust on the wall is  $96 \text{ kN/m}$  of wall. What will be the total active thrust if the water table comes up to backfill surface? Take specific gravity of backfill = 2.65.

(Engg. Services Exam. 2002)

**Solution :** (a) Water below the base of wall (Fig. 20.54 a)

$$P_{a1} = \frac{1}{2} K_a \gamma_d H^2 = \frac{1}{2} K_a (16) (6)^2 = 96 \text{ kN/m (given)}$$

$$\therefore K_a = \frac{96 \times 2}{16 (6)^2} = 0.333 = \frac{1}{3}$$

(b) Water at top surface (Fig. 20.54

(b)

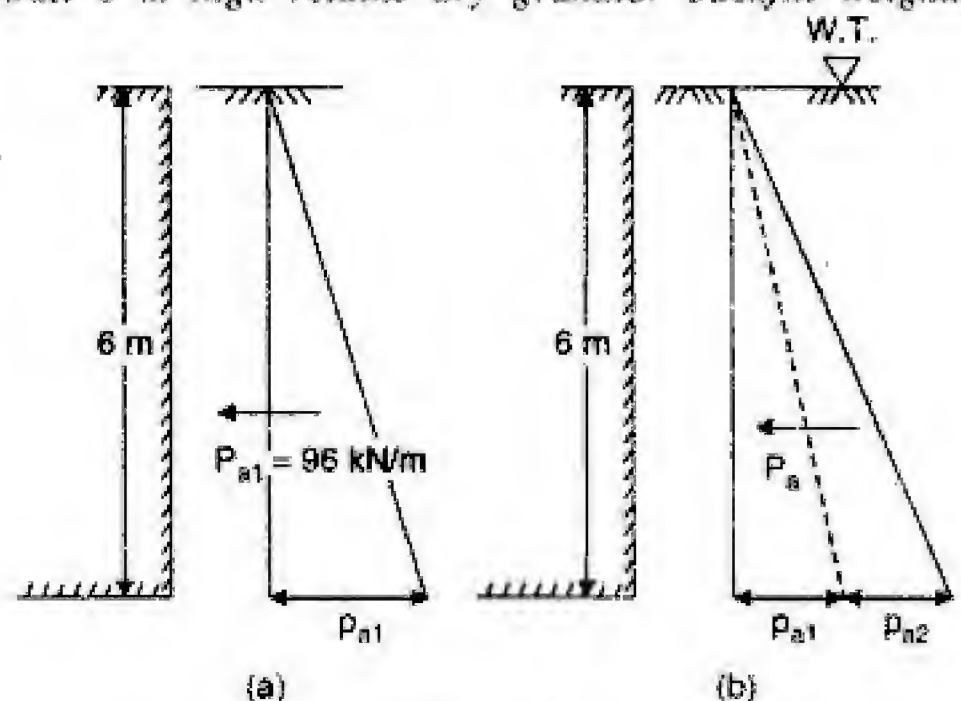
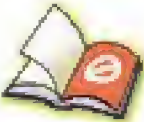


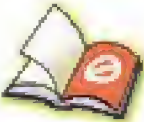
FIG. 20.54



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$$P_4 = \frac{1}{2} \times 30 \times 3 = 45 \text{ kN acting at } z_4 = 3/3 = 1 \text{ m above base}$$

∴ Total

$$P_1 = 8.672 + 26.016 + 12.195 + 45 = 91.883 \text{ kN}$$

$$\text{Acting at } z = \frac{8.672 \times 3.667 + 26.016 \times 1.5 + 12.195 \times 1 + 45 \times 1}{91.883} = 1.393 \text{ m above base.}$$

(b) *Stability analysis* (Fig. 20.59 c)

$$W_1 = 1 \times 5 \times 1 \times 24 = 120 \text{ kN/m} ; W_2 = \frac{1}{2} \times 2 \times 5 \times 24 = 120 \text{ kN}$$

$$\bar{x} = \frac{W_1 x_1 + W_2 x_2 + P_a \cdot z}{W_1 + W_2} = \frac{120 \times 0.5 + 120 \times 1.667 + 91.883 \times 1.393}{120 + 120} = 1.617 \text{ m}$$

$$e = \bar{x} - B/2 = 1.617 - 3/2 = 0.117 < B/6$$

$$(i) \text{ Factor of safety against sliding} = \frac{\mu \sum W}{\sum H} = \frac{0.47 \times 240}{91.883} = 1.23$$

$$(ii) \text{ Factor safety against overturning} = \frac{M_V}{M_H} = \frac{120 \times 0.5 + 120 \times 1.667}{91.883 \times 1.393} = 2.03$$

(iii) *Bearing pressure by Meyerhof's method :*

$$\text{Effective width } B' = B - 2e = 3 - 2 \times 0.117 = 2.766 \text{ m}$$

$$\therefore \text{Bearing pressure} = \frac{W_1 + W_2}{B' \times 1} = \frac{120 + 120}{2.766 \times 1} = 86.77 \text{ kN/m}^2$$

**Example 20.50.** A retaining wall with a stratified backfill and a surcharge load is shown in Fig. 20.60 (a). Draw the earth pressure diagram detailing the values at the critical points. Also estimate the resultant thrust on the wall and its position. (Gate Exam. 2002)

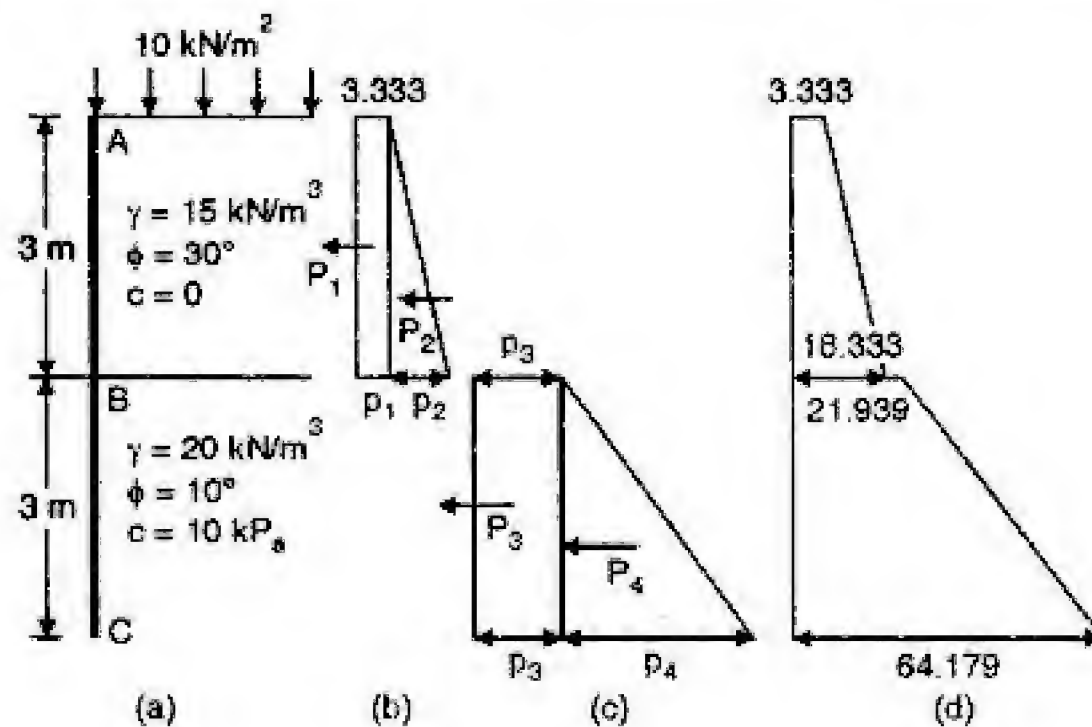


FIG. 20.60

**Solution :**

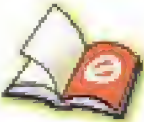
$$K_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

$$K_{a2} = \frac{1 - \sin 10^\circ}{1 + \sin 10^\circ} = 0.704$$

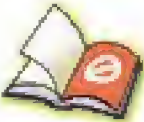


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according to which a bulkhead may be cantilever type or may be anchored. A *cantilever bulkhead* is the one which derives its stability entirely from the lateral resistance of the soil into which the sheet piling is driven [Fig. 21.1 (a)]. An *anchored bulkhead* is one which is held above its driven depth by one or more tie rods or anchors at or near its top [Fig. 21.1 (b)]. The stability of the cantilever bulkhead or sheet pile wall depends upon its embedment into soil below the dredged line. The stability of the anchored bulkhead depends more upon the anchors than on the inducement of large resultant earth pressure.

(c) *Classification according to sheet pile material.* Depending upon the material of sheet pile, a bulkhead may be classified as *flexible or rigid*. A *flexible bulkhead*, generally constructed of steel piles, deflects and bends under the influence of a lateral force. Its stability depends upon the development of sufficient lateral resistance of soil adjacent to the embedded length of the bulkhead, and upon the anchor rods. A *rigid bulkhead*, constructed of reinforced concrete sheet pile derives its stability against lateral forces basically from its inherent structural strength.

(d) *Classification according to condition of end support.* According to this, bulkheads may be classified into two categories: (i) bulkhead with free earth support, and (ii) bulkhead with fixed earth support. This classification applies only to anchored bulkheads. In the case of bulkhead with *free earth support*, the sheet piling is driven to a shallow depth only so that the deflection of the bulkhead is somewhat similar to that of a vertical elastic beam whose lower end is simply supported. In the case of bulkhead with *fixed earth support*, the sheet piles are driven to a considerable depth so that lower end of the bulkhead is practically fixed in position.

**Forces acting on a bulkhead.** A number of forces are known to be acting over a bulkhead, some of which are constantly in play while others come in effect only occasionally and intermittently. A sheet pile wall or bulkhead may be subjected to some or all of the following types of lateral pressures :

- (1) active and passive earth pressures, (2) lateral pressure due to surcharge load,
- (3) unbalanced water pressure and seepage pressure,
- (4) mooring pull, ship impact etc. (5) earth-quake forces.

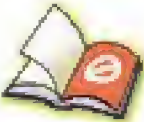
The classical earth pressure theories of Rankine or Coulomb, which are based on the condition of full mobilisation of the shear strength of soil due to yielding of the wall laterally by sliding or by rotation about its bottom, do not apply in the case of bulkheads where the deformation conditions are different. The earth pressure against sheet pile wall can be determined by theories which take into account the conditions of yield of the wall (Hansen, J.Brinch, 1953). In practice, several empirical and semi-empirical methods have been developed all of which use the classical earth pressure theories.

### 21.3. CANTILEVER SHEET PILE WALL

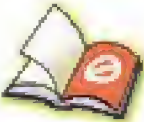
A cantilever sheet pile wall or bulkhead derives its stability entirely from the lateral resistance of the soil into which it is driven. The bulkhead is adequately embedded into the soil below the dredge line so that a driven line of sheeting acts as wide cantilever beam in resisting the lateral earth pressures developed above the dredge line. A cantilever bulkhead is used for moderate height only.



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$$p_a' = \left(\frac{1}{3} \times 3 \times 18\right) + \frac{1}{3} \times 3 (20 - 9.81)$$

$$= 18 + 10.19 = 28.19 \text{ kN/m}^2$$

$$a = \frac{p_a'}{\gamma' K} \quad (\text{Eq. 21.16}) = \frac{28.19}{10.19 \times 2.67} = 1.04 \text{ m}$$

$$R_a = \left(\frac{1}{2} \times 3 \times 18\right) + (18 \times 3) + \left(\frac{1}{2} \times 3 \times 10.19\right) + \left(\frac{1}{2} \times 28.19 \times 1.04\right) \\ = 27 + 54 + 15.3 + 14.7 = 111 \text{ kN}$$

$$y_1 = \frac{(27 \times 1) + (54 \times 3.5) + (15.3 \times 4) + (14.7 \times 5.35)}{111} = 3.2 \text{ m}$$

The depth  $Y$  is calculated from Eq. 21.20:

$$Y^3 \left( \frac{\gamma' K'}{3} \right) + Y^2 \left( \frac{\gamma' K'}{2} \right) (h + a) - R_a y_1 = 0$$

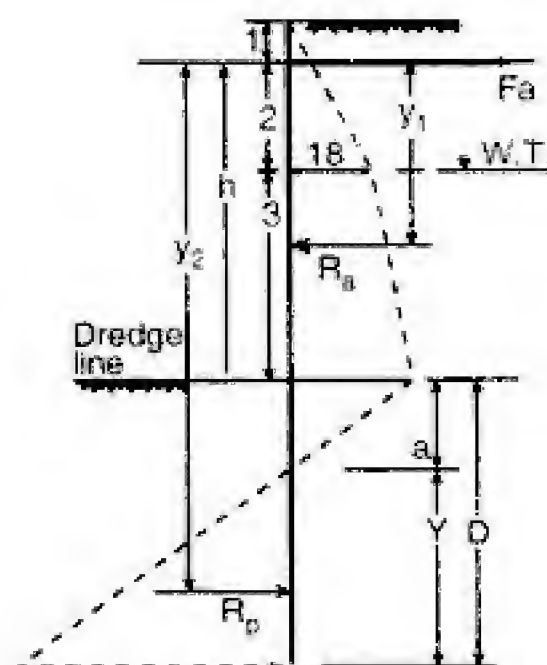


FIG. 21.15

$$\text{or} \quad Y^3 \left( \frac{10.19 \times 2.67}{3} \right) + Y^2 \left( \frac{10.19 \times 2.67}{2} \right) (5 + 1.09) - (111 \times 3.2) = 0$$

$$\text{or} \quad 9.07 Y^3 + 82.3 Y^2 - 355.2 = 0$$

$$\text{or} \quad Y^3 + 9.06 Y^2 = 39.2$$

By trial and error solution,  $Y = 1.9 \text{ m}$

$$\therefore D = Y + a = 1.9 + 1.04 = 2.94 \text{ m}$$

Providing 30% extra embedment,

$$D = 2.94 \times 1.3 = 3.82 \text{ m}$$

$$\text{Provide } D = 4 \text{ m,} \quad R_p = \frac{1}{2} Y^2 \gamma' K' \quad (\text{Eq. 21.18}) = \frac{1}{2} (1.9)^2 \times 10.19 \times 2.67 = 49.1 \text{ kN.}$$

$$\therefore F_a = R_a - R_p = 111 - 49.1 = 61.6 \text{ kN.}$$

**Example 21.4** Solve example 21.3 if the soil below the dredge line is purely cohesive, having the following properties :  $c = 50 \text{ kN/m}^2$  ;  $\gamma_{\text{sat}} = 19 \text{ kN/m}^3$  ;  $\phi = 0$ .

**Solution.** As before :  $K_a = K_a' = \frac{1}{3}$  ;  $K_p = K_p' = 3$

$$K' = 2.67, \quad \gamma' = 20 - 9.81 = 10.19$$

$$q \text{ (at the dredge line)} = (18 \times 3) + (10.19 \times 3) = 84.57 \text{ kN/m}^2$$

$$R_a = 111 \text{ kN (as before),} \quad y_1 = 3.2 \text{ m (as before)}$$

The depth  $D$  is determined from Eq. 21.24 :

$$D^2 + 2Dh - \frac{2y_1 R_a}{4c - q} = 0 \quad \text{or} \quad D^2 + (2 \times 5) D - \frac{2 \times 3.2 \times 111}{(4 \times 50) - 84.57} = 0$$

$$D^2 + 10D - 6.15 = 0, \quad \text{or} \quad D = 0.58 \text{ m}$$

Increasing  $D$  by 30%,  $D = 1.3 \times 0.58 = 0.76$

Hence provide  $D = 0.8 \text{ m}$



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$$\gamma = 21 \text{ kN/m}^3 ; \phi = 30^\circ ; c = 0$$

(b) Also determine the safe depth of embedment ( $d$ ) and tension in each tie rod. Also, design a continuous anchor to support the tie rods, if the tie rods are spaced 2 m centres.

(Civil Services Exam. 1997)

**Solution :**

$$K_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3} ; K_p = 1/K_a = 3.$$

$$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2} \times \frac{1}{3} \times 21 (5 + d)^2 = 3.5 (5 + d)^2 \text{ kN/m}$$

$$P_p = \frac{1}{2} K_p \gamma d^2 = \frac{1}{2} \times 3 \times 21 d^2 = 31.5 d^2 \text{ kN/m}$$

Taking moments about  $B$ ,

$$M_D = M_a = 3.5 (5 + d)^2 \left[ \frac{2}{3} (5 + d) - 1 \right] = 1.167 (5 + d)^2 [2d + 7]$$

$$M_R = M_p = 31.5 d^2 \left( 4 + \frac{2}{3} d \right) = 21 d^2 (6 + d)$$

Equating the two, we get  $1.167 (5 + d)^2 (7 + 2d) = 21 d^2 (6 + d)$

Solve the above equation by trial and error as under :

Trial No.	$d$	L.H.S.	R.H.S.
1.	1.6	518	408.5
2.	1.7	544.8	467.3
3.	1.8	572	530.7
4.	1.9	600	598.9

Hence  $d = 1.9 \text{ m}$ .

$$P_a = 3.5 (5 + 1.9)^2 = 166.4 \text{ kN/m} \text{ and } P_p = 31.5 (1.9)^2 = 113.72 \text{ kN/m}$$

$$\text{Force in tie rod} = T = P_a - P_p = 166.64 - 113.72 = 52.92 \text{ kN/m}$$

(b) **Safe Design :** In the above analysis, we have  $M_D = M_R$  i.e.  $F = 1$ . For safe design,  $F = 2$ , corresponding to which  $M_R = 2 M_D$

$$\therefore 21 d^2 (6 + d) = 2 \times 1.167 (5 + d)^2 (2d + 7)$$

$$\text{or } (5 + d)^2 (2d + 7) = 8.997 d^2 (6 + d)$$

Solving this by trial and error, we get  $d = 3.4 \text{ m}$ . Corresponding to this depth of embedment, we get

$$P_a = 3.5 (5 + 3.4)^2 = 246.96 \text{ kN/m}$$

$$P_p = \frac{1}{2} \times 31.5 (3.4)^2 = 182.07 \text{ kN/m, with a factor of safety of 2.}$$

$$\therefore T = P_a - P_p = 246.96 - 182.07 = 64.89 \text{ kN/m}$$

Since the tie rods, are spaced @ 2m c/c, force in each tie rod =  $2 \times 64.89 = 129.78 \text{ kN}$

$$\text{For continuous anchor, } s = L, \quad T = \frac{\gamma h_a^2}{2F} (K_p - K_a) \quad \text{or} \quad 64.89 = \frac{21 h_a^2 (3 - \frac{1}{3})}{2 \times 2}$$

$$\text{From which we get } h_a = 2.15 \text{ m}$$

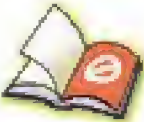
$$\text{Min. height of anchor is } b_{\min} = \frac{h_a}{2} = \frac{2.15}{2} \approx 1.08 \text{ m}$$

However, use continuous anchor of height 1.1 m

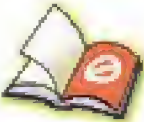
Also from scour point of view, keep  $d = 1.2 \times 3.4 \approx 4 \text{ m}$ .



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to the top level of the conduit. The trench is refilled with material placed in the loosest possible manner upto the top, and the rest of the embankment is constructed as usual. Marston recommended to use hay, straw, cornstalks etc. to be filled in trench, in order to increase the compressibility of this prism of soil and thus decrease the load on the conduit. This method of construction was used in Atlanta, Georgia, where the initial embankment of 35 ft, above a sewer constructed in 1937 was proposed to be raised to about 96 ft, in 1955. The sewer was protected from the increased load, by excavating a trench to a depth of 15 ft below natural ground, directly over the pipe. The trench was refilled with loose soil in which three 1 ft layers of tree leaves and pine straw were incorporated at approximately 5 ft intervals. Then as the embankment was constructed, shallow trenches were excavated over the sewer in each 15 ft lift of the compact fill. The depth of trench was about one half of the lift in which it was excavated. Then the trench was refilled with loose, uncompacted soil.

### 22.9. LOAD ON CONDUIT DUE TO LIVE LOADS

Marston proposed the following formula for computing additional load on the conduits due to loads applied to the surface of the fill.

$$W_t = \frac{1}{L_e} I_e C_t \cdot P \quad \dots(22.26)$$

where  $W_t$  = Average load on conduit, due to wheel load, (kN/m)  
 $L_e$  = effective length of conduit

(It is taken equal to the actual length of a precast segment section of pipe if less than 1 m. If the precast length is more than 1 m, the maximum value of  $L_e$  is recommended as 1 m)

$I_e$  = Impact factor

$C_t$  = Load co-efficient

$P$  = concentrated wheel load, on the embankment (kg).

The load coefficient  $C_t$  is dependent on the length and width of the conduit cross-section and the depth of cover over it may be evaluated in accordance with the Boussinesq formula of stress distribution.

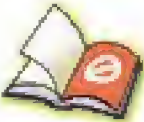


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$F$  equals to shear strength  $\tau_f$ . For any depth  $z$  less than that represented by point  $F$ , the shear stress  $\tau$  is less than the shear strength  $\tau_f$  and the slope remains stable. For example, the depth  $z$  corresponding to point  $C_1$  is stable for the slope of angle  $i > \phi$ . Hence, if  $i$  is greater than  $\phi$ , the slope can be stable only upto a limited depth known as the *critical depth*  $H_c$ .

The factor of safety against failure, for any depth  $z$  corresponding to point  $C_1$  of the slope angle  $i > \phi$ , is given by

$$F = \frac{\tau_f}{\tau} = \frac{c + \sigma \tan \phi}{\tau}$$

Putting  $\sigma = \gamma z \cos^2 i$   
and  $\tau = \gamma z \cos i \sin i$ , we get

$$\begin{aligned} F &= \frac{c + \gamma z \cos^2 i \tan \phi}{\gamma z \cos i \sin i} \\ &= \frac{c}{\gamma z \sin i \cos i} + \frac{\tan \phi}{\tan i} \end{aligned} \quad \dots(23.7)$$

Note that for non-cohesive soil,  $c = 0$  and hence Eq. 23.7 reduces to Eq. 23.6.

For the critical depth  $z = H_c$  corresponding to point  $F$ ,  $\tau_f$  equals  $\tau$  (i.e.  $F = 1$ ). Hence we get, from Eq. 23.7,

$$\gamma H_c \cos i \sin i = c + \gamma H_c \cos^2 i \tan \phi$$

$$\text{or} \quad H_c = \frac{c}{\gamma (\tan i - \tan \phi) \cos^2 i} \quad \dots(23.8)$$

Eq. 23.8 indicates that for given values of  $i$  and  $\phi$ ,  $H_c$  is proportional to cohesion.

$$\text{Rewriting Eq. 23.8,} \quad \frac{c}{\gamma H_c} = (\tan i - \tan \phi) \cos^2 i \quad \dots(23.9)$$

The dimensionless quantity  $\frac{c}{\gamma H_c}$  is called the *stability number*  $S_n$  :

$$S_n = \frac{c}{\gamma H_c} \quad \dots(23.10)$$

Let  $F_c$  represent the factor of safety with respect to cohesion, and let  $c_m$  be the mobilised cohesion, at depth  $H$ , given by

$$c_m = \frac{c}{F_c} \quad \dots(23.11)$$

The stability number is then written as

$$S_n = \frac{c}{\gamma H_c} = \frac{c_m}{\gamma H} = \frac{c}{F_c \gamma H} = (\tan i - \tan \phi) \cos^2 i \quad \dots(23.12)$$

From Eqs. 23.9 and 23.12, we get

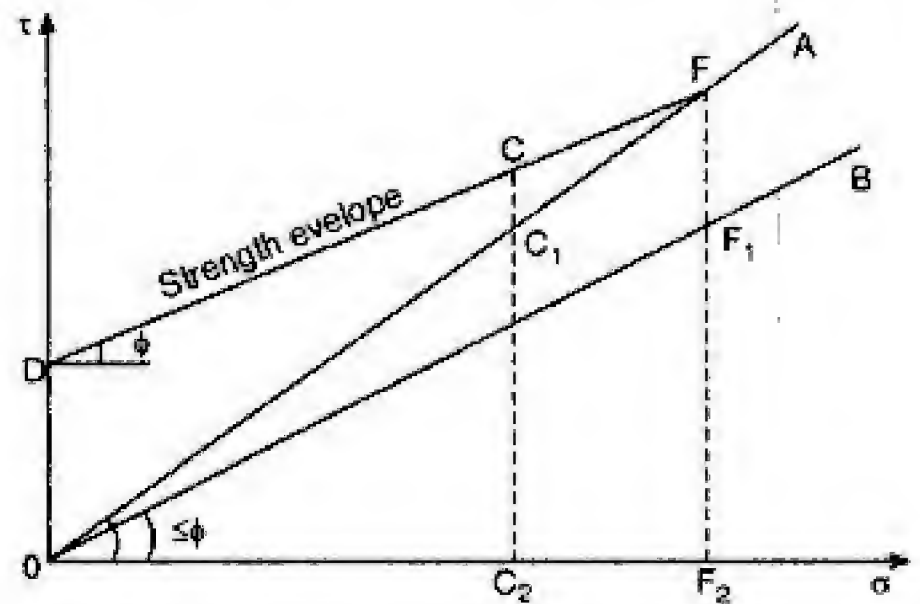
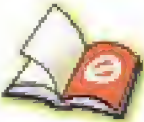
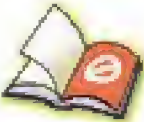


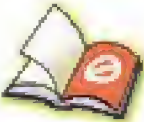
FIG. 23.4 FAILURE CONDITIONS OF AN INFINITE SLOPE OF COHESIVE SOIL



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TABLE 23.3. CONSOLIDATION TEST DATA

Consolidation $\Delta(\%)$	Effective stress $\sigma' \text{ (kg/cm}^2\text{)}$	Pore pressure $u \text{ (kg/cm}^2\text{)}$	Total stress $\sigma \text{ (kg/cm}^2\text{)}$
1	0.7	0.065	0.765
2	1.3	0.139	1.439
4	2.3	0.322	2.622
6	3.0	0.576	3.576
8	3.6	0.948	4.548
10	4.1	1.550	5.650
12	4.9	2.680	7.580
14	6.0	6.160	12.160
16	7.9	38.2	46.10

In order to find the construction pore pressure at the bottom of various slices( Fig. 23.8) the mid-heights  $z_1, z_2, \dots, z_n$  of the strips are measured and the total stresses  $\sigma$  (i . e .  $\sigma_1, \sigma_2, \dots, \sigma_n$ ) are calculated from the relation  $\sigma = \gamma z$ . Then from Fig.23.12 (b) the values of  $u$  corresponding to various values of  $\sigma$  are found and results tabulated as shown in Table 23.4.

TABLE 23.4. DETERMINATION OF PORE PRESSURE

Slice No.	Height $z$	Total stress $\sigma = \gamma z \text{ ((kg/cm}^2\text{))}$	Pore pressure	
			$u \text{ (kg/cm}^2\text{)}$	$U = u \times b \times l \text{ (kg)}$
			Sum :	$\Sigma U =$

Knowing  $\Sigma U$ , the factor of safety of the  $u/s$  or  $d/s$  slope is calculated by the

equation 
$$F = \frac{c' \hat{L} + \tan \phi' \Sigma(N - U)}{\Sigma T}$$

**Example 23.4.** In order to find the factor of safety of  $d/s$  slope of an earth dam, during steady seepage, the section of the dam was drawn to a scale of 1 cm = 4 cm, and the following results obtained on a critical slip circle:

Area of N-rectangle = 14.4 sq. cm.

Area of T-rectangle = 6.4 sq.cm.

Area of U-rectangle = 6.9 sq. cm

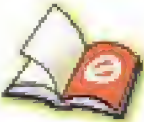
Length of arc = 12.6 cm.

Laboratory tests have furnished values of  $26^\circ$  for effective angle of shear resistance and  $19.5 \text{ kN/m}^2$  for cohesion. Determine the factor of safety of the slope. Unit wt. of soil =  $19 \text{ kN/m}^3$ .



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Now  $S_n = \frac{c}{F \cdot \gamma H} \quad \therefore \quad H = \frac{c}{S_n F \cdot \gamma} = \frac{15}{0.046 \times 1.5 \times 19} \approx 11.5 \text{ m.}$

**Example 23.7.** A new canal is excavated to a depth of 5 m below ground level, through a soil having the following characteristics:  $c = 14 \text{ kN/m}^2$ ;  $\phi = 15^\circ$ ;  $e = 0.8$  and  $G = 2.70$ . The slope of banks is 1 in 1. Calculate the factor of safety with respect to cohesion when the canal runs full. If it is suddenly and completely emptied, what will be the factor of safety ?

**Solution**

$$\gamma_{sat} = \frac{G + e}{1 + e} \gamma_w = \frac{2.7 + 0.8}{1 + 0.8} 9.81 = 19.08 \text{ kN/m}^3$$

$$\gamma' = 19.08 - 9.81 = 9.27 \text{ kN/m}^3$$

$$i = 45^\circ; \phi = 15^\circ$$

(i) **Submerged case :** For  $i = 45^\circ$ ;  $\phi = 15^\circ$ ;  $S_n = 0.083$

$$\therefore F_c = \frac{c}{\gamma' H S_n} = \frac{14}{9.27 \times 5 \times 0.083} = 3.64$$

(ii) **Drawdown case :** Taking  $F_\phi = 1$  and  $\phi_m = \phi$

$$\phi_w = \frac{\gamma'}{\gamma_{sat}} \phi_m = \frac{\gamma'}{\gamma_{sat}} \phi = \frac{9.27}{19.08} \times 15 \approx 7.3^\circ$$

$$\text{For } i = 45^\circ \text{ and } \phi = 7.3^\circ, S_n = 0.122$$

$$\therefore F_c = \frac{c}{\gamma_{sat} H S_n} = \frac{14}{19.08 \times 5 \times 0.122} = 1.2$$

**Example 23.8.** A temporary cutting 8 m deep is to be made in a clay having a unit weight of  $18 \text{ kN/m}^3$  and an average cohesion of  $20 \text{ kN/m}^2$ . A hard stratum of rock exists at a depth of 12 m below the ground surface. Use Taylor's stability curves to estimate if a  $30^\circ$  slope is safe. If a factor of safety of 1.25 is considered necessary, find the safe slope angle.

**Solution :** Depth factor  $D_f = \frac{12}{8} = 1.5$ .

From Taylor's stability curves for  $D_f = 1.5$ ,  $i = 30^\circ$ ,  $S_n = 0.163$

$$\therefore F_c = \frac{c}{S_n \gamma H} = \frac{20}{0.163 \times 18 \times 8} = 0.85$$

The proposed slope is therefore, unsafe. For  $F_c = 1.25$ , the required stability number is

$$S_n = \frac{c}{F_c \gamma H} = \frac{20}{1.25 \times 18 \times 8} = 0.11$$

Form Taylors curves, for  $S_n = 0.11$  and  $D_f = 1.5$  we get  $i = 12^\circ$ .

**Example 23.9** A slope is 12 m high, and has an inclination of  $30^\circ$ . If the soil of the slope has  $c = 25 \text{ kN/m}^2$ ,  $\phi = 12^\circ$  and  $\gamma = 18.6 \text{ kN/m}^3$ , determine the factor of safety with respect to shear strength.

**Solution** The factor of safety  $F (= F_c = F_\phi)$  is found by trial.



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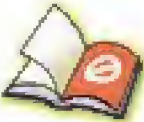


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- (ii) There is considerable bulging of sheared mass of soil adjacent to the footing
- (iii) Failure is accompanied by tilting of the footing
- (iv) Failure is sudden, with pronounced peak resistance
- (v) The ultimate bearing capacity is well defined

2. **Local shear failure** : In *local shear failure*, there is significant compression of the soil under the footing and only partial development of state of plastic equilibrium. Due to this reason, the failure surfaces do not reach the ground surface and only *slight* heaving occurs. The pressure-settlement curve is represented by curve *b* of Fig. 24.2(d), where the peak of the base resistance may never be reached. In such a failure, tilting of foundation is not expected. Local shear failure is associated with soils of high compressibility and in sands having relative density lying between 35 and 70 percent. The failure is not sudden, and it is characterised by occurrence of relatively large settlements which would not be acceptable in practice. Also, ultimate bearing capacity in such a failure is not well defined. Following are typical *characteristics* of local shear failure :

1. Failure pattern is clearly defined only immediately below the footing
2. The failure surfaces do not reach ground surface
3. There is only slight bulging of soil around the footing
4. Failure is not sudden and there is no tilting of footing
5. Failure is defined by large settlements
6. Ultimate bearing capacity is not well defined.

3. **Punching shear failure** : *Punching shear failure* occurs where there is relatively high compression of soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. Punching shear may occur in relatively loose sand with relative density less than 35%. Punching shear failure may also occur in a soil of low compressibility if the foundation is located at considerable depth. The failure surface, which is vertical or slightly inclined and follows the perimeter of the base, never reaches the ground surface. There is no heaving of the ground surface away from the edges and no tilting of the footing. Relatively large settlements occur in this mode. The ultimate bearing capacity is not well defined. Following the *characteristics* of punching shear failure:

- (i) No failure pattern is observed.
- (ii) The failure surface, which is vertical or slightly inclined, follows the perimeter of the base.
- (iii) There is no bulging of soil around the footing
- (iv) There is no tilting of footing
- (v) Failure is characterised in terms of very large settlements
- (vi) The ultimate bearing capacity is not well defined.

**Conditions for typical mode of failure :**

Table 24.1 gives the conditions under which a typical mode of failure may occur.

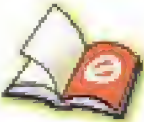
#### **Modes of failure of footings in sand**

Fig. 24.3 shows the modes of failure of footings in sand. As the relative depth/width ratio increases, the limiting relative densities at which failure type change increases.



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In Eq. 24.34,

$\gamma_1$  = average unit weight of the surcharge soil situated above the water table.

$\gamma_2$  = average unit weight of the soil in the wedge zone, situated within a depth  $B$  below the base of the footing.

When the water table is just at the base of the footing,  $\gamma_2 = \gamma_{sat}$ .

When the water table is at the ground surface, both  $\gamma_1$  and  $\gamma_2$  are the saturated weights. For intermediate locations of water table,  $\gamma_1$  and  $\gamma_2$  can be computed, using rule of three as illustrated in Example 24.4.

(ii) **Second Method : IS Code Method** : Alternatively,  $q_f$  can be computed using only one reduction factor, by rewriting Eq. 24.34 as under:

$$q_f = cN_c + \bar{\sigma}N_q + \frac{1}{2}\gamma BN_\gamma R_w$$

where

$\bar{\sigma}$  = effective unit weight of the soil situated above the base

$$R_w = R_{w2} = 0.5 \left( 1 + \frac{Z_{w2}}{B} \right) \quad \dots(24.36)$$

Eq. 24.36 has been recommended by Indian Standard. When the water table is situated at a depth  $D_1$  below the ground level ( $D_1 < D$ ) or  $D_2$  above the base of the footing (such that  $D_1 + D_2 = D$ ), we have  $\bar{\sigma} = (\gamma D_1 + \gamma_{sat} D_2) - \gamma_w \cdot D_2 = \gamma D_1 + \gamma' D_2 \quad \dots(24.36 \text{ a})$

Knowing  $\bar{\sigma}$  and  $R_w (= R_{w2})$ ,  $q_f$  can be computed.

This alternative method is preferred to the method of Eq. 24.34 in which two reduction factors  $R_{w1}$  and  $R_{w2}$  are used. See Example 24.4 for illustration and comparison.

(iii) **Third Method**. In the third method (Bowles, 1988), no water reduction factor is used, but effective unit weight ( $\gamma_e$ ) is used for the soil in the wedge zone.

Thus, we have  $q_f = cN_c + \bar{\sigma}N_q + \frac{1}{2}\gamma_e BN_\gamma \quad \dots(24.37 \text{ a})$

where  $\bar{\sigma}$  = effective overburden pressure

and  $\gamma_e$  = effective unit weight of soil in the wedge zone.

The wedge zone has a depth  $H = 0.5 B \tan (45^\circ + \phi/2)$ . Hence when the water table is within this wedge zone,  $\gamma_e$  can be computed from the expression

$$\gamma_e = (2H - z_{w2}) \frac{z_{w2}}{H^2} \gamma + \frac{\gamma'}{H^2} (H - z_{w2})^2 \quad \dots(24.37 \text{ b})$$

See example 24.6. for illustration

**Alternatively**, taking  $H = B$ , the effective unit weight  $\gamma_e$  in the wedge term can be

computed as under :  $\gamma_e = \frac{\gamma z_{w2} + \gamma' (B - z_{w2})}{B} = \gamma' + (\gamma - \gamma') \frac{z_{w2}}{B} \quad \dots(24.37 \text{ c})$

when  $z_{w2} = 0$ ,  $\gamma_e = \gamma'$  (as expected)

when  $z_{w2} = B$ ,  $\gamma_e = \gamma$  (as expected). See example 24.6. for illustration.

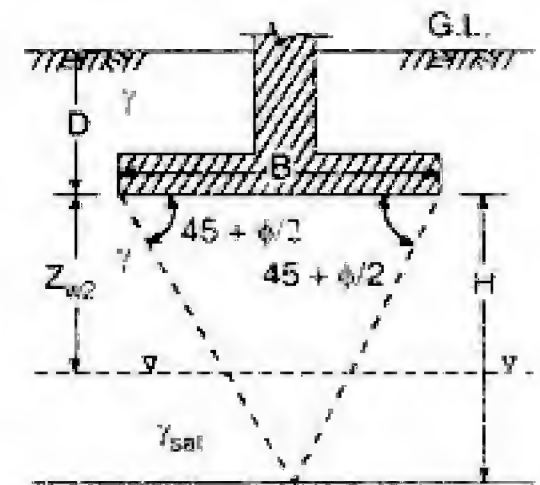


FIG. 24.11



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tlement axis as the settlement increases. For partially cohesive soils, curve *C* possessing the characteristics of both curves *A* and *B* is obtained, while curve *D* is purely for dense cohesionless soils. For curves *B* and *D*, no difficulty will be experienced in arising at the ultimate bearing capacity as the failure is well defined. However, in cases of curves *A* and *C*, where yield point is not well defined, settlements are plotted as abscissa against corresponding load intensities as ordinate, both to logarithmic scale (Fig. 24.25). Such a plotting will give two straight lines, the intersection of which will be considered as yield value of soil.

IS : 1888 : 1982 has not specified any factor of safety. In order to determine the safe bearing capacity, it would be normally sufficient to use a factor of safety of 2 or 2.5 on the ultimate bearing capacity.

**Limitations of Plate Load Test :** The plate loading test has the following *limitations*:

1. The test results reflect only the character of the soil located within a depth less than twice the width of the bearing plate (corresponding to an isobar of one-tenth the loading intensity at the test plate). Since the foundations are generally larger, the settlement and resistance against shear failure will depend on the properties of a much thicker stratum.

2. It is essentially a short duration test, and hence the test does not give the ultimate settlement, particularly in the case of cohesive soil.

3. Another limitation is the effect of the size of foundation. For clayey soils the ultimate pressure for a large foundation is the same as that for the test plate. But in dense sandy soils, the bearing capacity increases, with the size of the foundation, and the test on smaller size bearing plates tend to give conservative values.

**Effect of the size of plate on bearing capacity :** As stated in limitation 3 above, the bearing capacity of sands and gravels increases with the size of the footing. The relationship can be expressed as under :

$$q_F = M + N \frac{B_F}{B_P} \quad \dots(24.48)$$

In the above relation, *M* term includes the  $N_c$  and  $N_q$  terms while *N* includes  $N_\gamma$  portion of Eqs. 24.12 to 24.14.

Eq. 24.48 can be solved graphically by using more than one size plates. By extrapolating the plate load test data, one may use the following equation for all practical purposes:

$$q_F = q_P \cdot \frac{B_F}{B_P} \quad \dots(24.49)$$

where  $q_F$  = bearing capacity of the actual footing ;  $B_F$  = width of actual footing.

$q_P$  = bearing capacity, obtained from plate load test

$B_P$  = width of plate.

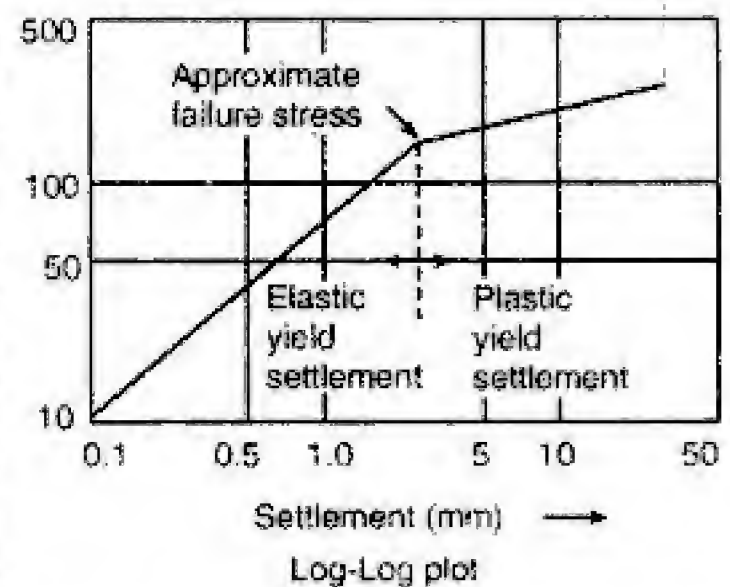
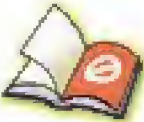


FIG 24.25 LOAD SETTLEMENT CURVE ON LOG-LOG PLOT



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Table 24.14 and 24.15 give some empirical correlations of the soil properties with corrected penetration resistance. The approximate values of  $\phi$  are after Meyerhoff (1956).

TABLE 24.14. PENETRATION RESISTANCE AND EMPIRICAL CORRELATIONS FOR COHESIONLESS SOILS

Penetration resistance $N$ (blows)	Approx. $\phi$ (degrees)	Density Index (%)	Description	Approx. moist unit weight ( $kN/m^3$ )
—	25–30	0		
4	27–32	15	Very Loose	1.12–1.6
10	30–35	35	Loose	1.44–1.84
30	35–40	65	Medium	1.76–2.08
50	38–43	85	Dense	1.76–2.24
—		100	Very dense	2.08–2.40

Also  $\phi = 25 + 0.15 I_D$ , with fines greater than 5%.

$\phi = 30 + 0.15 I_D$ , with fines less than 5%.

Larger values should be used for granular soil with 5% or less fine sand and silt.

TABLE 24.15. PENETRATION RESISTANCE AND EMPIRICAL CORRELATION FOR COHESIVE SOILS

Penetration resistance (blows)	Unconfined compressive strength ( $t/m^2$ )	Saturated density ( $t/m^3$ )	Consistency
0	0	—	Very soft
2	2.5	1.6–1.92	Soft
4	5		Medium
8	10	1.76–2.08	Stiff
16	20	1.92–2.24	Very stiff
32	40		Hard

## 2. STATIC CONE TEST : IS 4968 (Part 3) : 1976/1987

*Dutch static cone penetration test*, commonly abbreviated as CPT is used for getting a continuous record of the resistance of soil by penetrating steadily under static pressure a cone with a base of  $10 \text{ cm}^2$  (3.6 cm in dia.) and an angle of  $60^\circ$  at the vertex. The cone is carried at the lower end of a steel driving rod which passes through a steel tube (mantle) with external diameter equal to the base of the cone. Either the cone or the tube, or both together can be forced into the soil by means of jacks. To know the cone resistance, the cone alone is first forced down for a distance of 8 cm and the maximum value of resistance is recorded. The steel tube is then pushed down upto the cone, and both together are further penetrated through a depth of 20 cm to give the total of the cone resistance and the frictional resistance along the tube.

**Normalisation of  $q_c$  values :** Similar to SPT values ( $N$ ), the CPT values ( $q_c$ ) also require *normalisation* to the standard overburden pressure of  $100 \text{ kN/m}^2$ , using the relation

$$q_{cr} = C_n \cdot q_c \quad \dots(24.56 \text{ a})$$



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**I.S. code equation :** IS 6403 - 1971 accepted Teng's equation for 40 mm settlement, without depth factor  $R_d$ , expressed as follows :

$$q_p = 55 (N - 3) \left( \frac{B + 0.3}{2 B} \right)^2 R_w \quad \dots (24.61 \ a)$$

However, IS 6403-1981 dropped this equation.

**(b) Use of cone penetration resistance ( $q_c$ ) values**

Meyerhof (1956) suggested the following simplified formulae for determining allowable bearing pressure for a permissible settlement of 25 mm for foundations on *dry* sands:

**(i) Square or strip foundation equal to or less than 1.2 m wide**

$$q_p = 3.6 q_c \text{ kN/m}^2 \quad \dots (24.64 \ a)$$

**(ii) Square or strip footing greater than 1.2 m wide**

$$q_p = 2.1 q_c \left( 1 + \frac{1}{B} \right)^2 \text{ kN/m}^2 \quad \dots (24.64 \ b)$$

An approximate formula to cover all foundations, irrespective of width is

$$q_p = 2.7 q_c \text{ kN/m}^2 \quad \dots (24.64 \ c)$$

In all the above formulae,  $q_c$  is in  $\text{kg/cm}^2$ . The above formulae are based on the assumption that  $N$ -value is one-quarter of static cone resistance (in  $\text{kg/cm}^2$ ). However, Fig. 24.23 shows the relationship between static cone ( $q_c$ ) resistance and  $N$  (Thornburn, 1971), where  $q_c$  is in  $\text{kg/cm}^2$ .

The value of  $q_p$  in Eq. 24.62 should be *halved* if the sand within the stressed zone is submerged.

**IS code method :** Fig. 24.15 gives the ratio of  $q_{nf}/q_c$  for various values of  $D/B$  ratio and width  $B$  as recommended by IS code (IS : 6403—1981). From these curves, the net ultimate bearing capacity can be determined corresponding to a known value of  $q_c$ .

## 24.16. PERMISSIBLE TOTAL AND DIFFERENTIAL SETTLEMENTS

The vertical downward movement of the base of a structure is called settlement and its effect upon the structure depends on its magnitude, its uniformity, the length of the time over which it takes place and the nature of the structure itself.

It is suggested that allowable pressure should be selected such that maximum settlement of any individual foundation is 25 mm. It has also been suggested that differential settlement of uniformly loaded continuous foundation and of equally loaded spread foundation of approximately the same size, is unlikely to exceed half the maximum settlement, and that normal structures such as office buildings and flats can satisfactorily withstand differential settlements of about 18 mm between adjacent columns spaced 6 to 8 m apart.

According to National Building Code of India (SP : 7-1970), the differential settlement shall be kept within limits to which the super-structure can accommodate itself without harmful distortion, by suitably designing the foundation. Total settlements shall be so restricted or



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$$\therefore \text{Additional safe load capacity} = Q_{s2} = Q_{f2}/F = 200 B/4 = 50 B$$

$$\therefore \text{Total safe load capacity, } Q_s = Q_{s1} + Q_{s2} = 295.75 B^2 + 50 B \quad \dots(3)$$

$$\text{Actual load} = 3250 + 3.5 B^2 \times 25 = 3250 + 87.5 B^2 \quad \dots(4)$$

$$\text{Equating (3) and (4), } 295.75 B^2 + 50 B^2 = 3250 + 87.5 B^2$$

$$\text{or} \quad 208.5 B^2 + 50 B - 3250 = 0$$

From which  $B = 3.83 \text{ m.}$  Hence provide  $3.85 \text{ m} \times 3.85 \text{ m}$  size square footing.

**Example 24.26.** A strip footing is needed to carry a load of  $1000 \text{ kN/m}$  at a depth of  $1 \text{ m}$ . Shear strength parameters for the soil are  $c = 0$  and  $\phi = 36^\circ$ . Determine the minimum width of footing for a factor of safety of 3 against shear failure. The water table may rise to the base of the footing. Take  $G_s = 2.65$ ,  $e = 0.6$  and  $\gamma = 16 \text{ kN/m}^3$ . (Civil Services Exam, 1994)

$$\text{Solution :} \quad \gamma_{\text{sat}} = \frac{G + e}{1 + e} \gamma_w = \frac{2.65 + 0.6}{1 + 0.6} \times 9.81 = 19.93 \text{ kN/m}^3$$

$$\therefore \quad \gamma' = 19.93 - 9.81 = 10.12 \text{ kN/m}^3$$

$$\text{Now} \quad q_{nf} = c N_c + \bar{\sigma} (N_q - 1) + \frac{1}{2} B \gamma' N_\gamma$$

Since the water table is at the base of the footing,  $\bar{\sigma} = \gamma D$ , where  $\gamma$  is the wet weight, while  $\gamma$  will be taken as  $\gamma'$  with the wedge term.

$$\therefore \quad q_{nf} = c N_c + \gamma D (N_q - 1) + \frac{1}{2} B \gamma' N_\gamma \quad \dots(1)$$

Let us use IS Code method, which is based on Vesic's equation

$$\therefore \quad N_q = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) e^{z \tan \phi} = \tan^2 \left( 45^\circ + \frac{36^\circ}{2} \right) e^{z \tan 36^\circ} = 37.75$$

$$N_c = (N_q - 1) \cot \phi = (37.75 - 1) \cot 36^\circ = 50.58$$

$$N_\gamma = 2 (N_q + 1) \tan \phi = 2 (37.75 + 1) \tan 36^\circ = 56.31$$

$$\therefore \quad q_{nf} = 0 + 16 \times 1 (37.75 - 1) + \frac{1}{2} B \times 10.12 \times 56.31 = 588 + 284.9 B$$

$$q_s = \frac{q_{nf}}{F} + \gamma' \cdot D = \frac{588 + 284.9 B}{3} + 16 \times 1 = 212 + 95 B \quad \dots(2)$$

$$\text{Actual load intensity, } q = \frac{1000}{B \times 1} = \frac{1000}{B} \text{ m}^2 \quad \dots(3)$$

$$\therefore \quad \frac{1000}{B} = 212 + 95 B \quad \text{or} \quad B^2 + 2.23 B - 10.53 = 0$$

From which  $B \approx 2.32 \text{ m}$

**Example 24.27.** Explain briefly how the bearing capacity is determined by field load test. The result of two field tests on a given location are as follows:

(i) Diameter =  $750 \text{ mm}$  ;  $S = 15 \text{ mm}$  ;  $Q = 150 \text{ kN}$

(ii) Diameter =  $300 \text{ mm}$  ;  $S = 15 \text{ mm}$  ;  $Q = 50 \text{ kN}$

Determine the load on a circular footing of  $1.2 \text{ m}$  diameter that will cause a settlement of  $15 \text{ mm}$ . (Civil Services Exam, 1995)

**Solution :** Housel (1992) equation :  $Q = A \cdot q + P \cdot s$

$$\therefore \quad Q_1 = 150 = \frac{\pi}{4} (0.75)^2 q + \pi (0.75) \cdot s$$

$$\text{or} \quad 0.4418 q + 2.3562 s = 150 \quad \dots(1)$$

$$\text{and} \quad Q_2 = 50 = \frac{\pi}{4} (0.3)^2 q + \pi (0.3) s$$



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$$\therefore q_s = \frac{q_{nf}}{F} + \gamma' D = \frac{34.5}{3} + 0.817 \times 1.5$$

$$\approx 12.726 \text{ kN/m}^2$$

$$\therefore Q_s = q_s \times \text{Area} = 12.726 \times 2 \times 2 = 50.9 \text{ t}$$

(c) **Determination of settlement** : Consider level AA at mid depth of clay.

$$\sigma'_0 = \gamma'(2.5) = 0.817 \times 2.5 = 2.043 \text{ t/m}^2$$

$$\text{Area of spread at level AA} = 3 \times 3 = 9 \text{ m}^2$$

$$\therefore \Delta \sigma' = \frac{Q_s}{A} = \frac{50.9}{9} = 5.656 \text{ t/m}^2$$

$$C_c = 0.009 (w_L - 10) = 0.009 (30 - 10)$$

$$= 0.18 ; \quad e_0 = e = 1.08$$

$$\therefore \Delta H = \frac{C_c \cdot H}{1 + e_0} \log_{10} \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} = \frac{0.18 \times 2}{1 + 1.08} \log_{10} \frac{2.043 + 5.656}{2.043} \approx 0.1 \text{ m} = 10 \text{ cm}$$

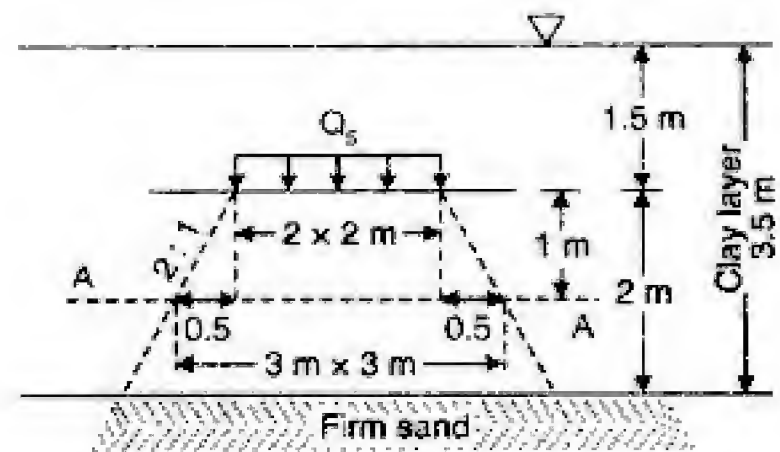


FIG. 24.34

**Example 24.38.** The following data was obtained from plate load test carried out on a 60 cm square test plate at a depth of 2 m below ground surface on a sandy soil which extends upto a large depth.

Load intensity ( $\text{t/m}^2$ )	Settlement (mm)	Load intensity ( $\text{t/m}^2$ )	Settlement (mm)
0	0	25	16.3
5	2.0	30	23.5
10	4.0	35	34.0
15	7.5	40	45.0
20	11.0		

Determine the settlement of foundation  $3.0 \text{ m} \times 3.0 \text{ m}$  carrying a load of  $110 \text{ t}$  and located at a depth of  $2 \text{ m}$  below ground surface. Water table is located at a large depth from the ground surface.

(Engg. Services Exam. 1996).

**Solution** : Fig. 24.35 shows the plot between load intensity and settlement.

Actual load intensity

$$= \frac{110}{3 \times 3} = 12.22 \text{ t/m}^2$$

Hence from the graph,

$$\rho_F = 4.5 \text{ mm}$$

Now

$$\rho_F = \rho_P \left[ \frac{B (B_P + 0.3)}{B_P (B + 0.3)} \right]^2 = 4.5 \left[ \frac{3 (0.6 + 0.3)}{0.3 (3 + 0.3)} \right]^2 \approx 33.5 \text{ mm}$$

**Example 24.39.** A  $3.0 \text{ m}$  square footing is located in a dense sand at a depth of  $2.0 \text{ m}$ . Determine the ultimate bearing capacity for the following water table positions:

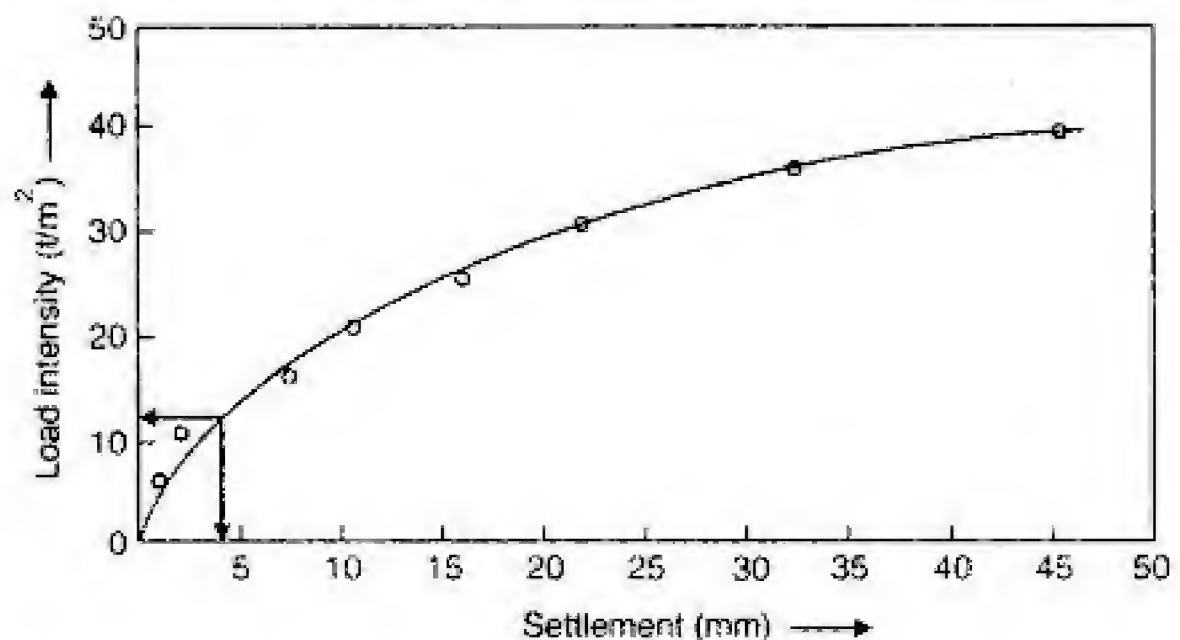
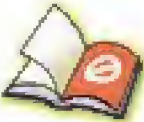
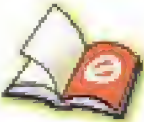


FIG. 24.35



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$I_w$  = Influence factor = 0.88 for rigid circular footing  
 = 0.82 for rigid square footing  
 = 1.06 for rigid rectangular footing with  $L/B = 1.5$   
 = 1.70 for rigid rectangular footing, with  $L/B = 5$

Table 25.1 gives the values of *influence factor*  $I_w$  as suggested by BIS Code IS : 8009 Part I, 1976. According to this code, the total settlement of a rigid footing is taken as 0.8 times the settlement at the center of a flexible foundation. Accordingly, column 5 of Table 25.1 gives the values of  $I_w$  by multiplying the values of column (2) by 0.8.

TABLE 25.1  
VALUES OF INFLUENCE FACTOR  $I_w$  FOR USE IN EQ 25.9 (IS : 8009 PART I-1976)

Shape of loaded area	Flexible Footing			Rigid Footing Col. (5) = 0.8 × Col. (2)
	Centre	Corner	Average	
(1)	(2)	(3)	(4)	(5)
1. Circular	1.00	0.64	0.85	0.80
2. Rectangular ( $L/B$ )				
1.0	1.12	0.56	0.95	0.90
1.5	1.36	0.68	1.20	1.09
2.0	1.52	0.77	1.31	1.22
5.0	2.10	1.05	1.83	1.68
10.0	2.52	1.26	2.25	2.02
100.0	3.38	1.69	2.96	2.70

It is difficult to determine  $\mu$  and  $E_s$  for soils. However, the entire term  $\frac{1 - \mu^2}{E_s} I_w$  for Eq. 25.6 may be determined from plate load test by using two or three different size plates of the same shape. A plot between  $S_i$  and  $q \cdot B$  gives a straight line, the slope of which is equal to  $\frac{1 - \mu^2}{E_s} I_w$ .

Janbu, Bjerrum and Kjaernsli (1966) have proposed the following equation for computing the immediate settlement

$$S_i = \mu_0 \mu_1 q B \frac{1 - \mu^2}{E_s} \quad \dots(25.10)$$

where the modulus of elasticity  $E_s$  is computed from triaxial test data

$$E_s = \frac{\sigma_1 - \sigma_3}{\Delta L/L} \quad \dots(25.11)$$

and  $\mu_0$  and  $\mu_1$  are taken from Fig. 25.5.

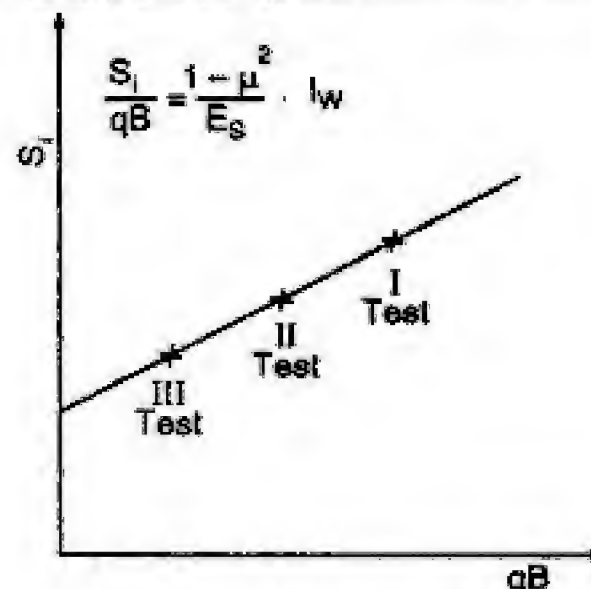
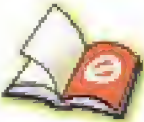
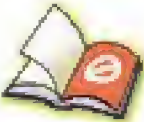


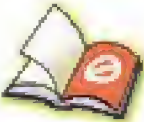
FIG 25.4 GRAPHICAL SOLUTION TO FIND  $\frac{1 - \mu^2}{E_s} I_w$



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$$q_p = 2 (N-3) R_{w2} (t/m^2) \approx 20(N-3) R_{w2} \text{ kN/m}^2 \quad \dots(25.23)$$

The penetration resistance  $N$  values should be taken at 75 cm intervals for depths equal to width of the raft, below the base of the raft. The minimum average value of  $N$  for the holes should be used in Eq. 25.23. If  $N$  is less than 5, sand should be compacted by artificial means to raise  $N$  above 10, or else piles or piers should be used.  $N$  should be corrected for fine sand below water table.

## 25.7. I.S. CODE OF PRACTICE FOR DESIGN OF RAFT FOUNDATIONS

IS : 2950-1965 gives the code of practice for design and construction of raft foundations. The maximum differential settlement should not exceed 40 mm in foundation on clayey soils and 25 mm in foundations on sandy soils. The maximum settlement should generally be limited to the following values :

Raft foundation on clay : 65 to 100 mm.

Raft foundation on sand : 40 to 65 mm.

The behaviour of a raft foundation being complicated, a number of simplifying assumptions have to be made in the design. There are two approaches for design—*conventional method* and the *elastic method or the soil line method*.

1. **Conventional method** : The conventional method is based on the following two basic assumptions :

(i) The foundation is infinitely rigid and, therefore, the actual deflection of the raft does not influence the pressure distribution below the raft.

(ii) The soil pressure is assumed to be planer such that the centroid of the soil pressure coincides with the line of action of the resultant force of all the loads acting on the foundation.

In the conventional method, the allowable bearing pressure is found using the following formulae :

$$q_1 = 21.4 N^2 B R_{w1} + 64(100 + N^2) D R_{w2} \quad \dots(25.24 \text{ a})$$

$$q_2 = 1950 (N - 3) R_{w2} \quad \dots(25.24 \text{ b})$$

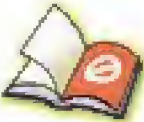
where  $q_1$  and  $q_2$  = allowable soil pressure under raft foundation in  $\text{kg/m}^2$   
(using a factor of safety of three)

$R_{w1}$  and  $R_{w2}$  = reduction factors on account of sub-soil water as explained in chapter 24.

The smaller of the two values of  $q_1$  and  $q_2$  should be used for design. The effect of overburden pressure on the  $N$  value should also be considered as explained in chapter 24. In the case of saturated silts, the equivalent penetration resistance  $N_e$ , for the values of  $N$  greater than 15 should be taken for design as explained in chapter 24.

The pressure distribution ( $q$ ) under the raft should be determined by the following formula :

$$q = \frac{Q}{A} \pm Q \frac{e_y'}{I_x'} y \pm Q \frac{e_x'}{I_y'} x \quad \dots(25.25)$$

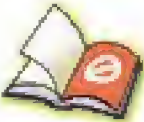


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$$\text{But } \bar{x} = \frac{L}{3} \left( \frac{2a+b}{a+b} \right) \quad \therefore \quad \frac{2a+b}{a+b} = \frac{3}{6.8} \times 2.65 = 1.169$$

$$\text{or } 0.831a - 0.169b = 0$$

$$\text{or } b = 4.917a$$

Substituting this values in (1), we get  $a = \frac{5.882}{5.917} = 0.994 \text{ m}$  and  $b = 4.889 \text{ m}$

Hence use trapezoidal footing of size  $a = 1 \text{ m}$ ;  $b = 4.9 \text{ m}$  and  $L = 6.4 \text{ m}$

**Example 25.8.** A plan of a proposed heap is shown in Fig. 25.14 (a) The heap will stand on a thick deposit of soft clay with  $E$ -value of  $15 \text{ MN/m}^2$ . The uniform pressure on the soil may be assumed as  $150 \text{ kN/m}^2$ . Estimate the immediate settlement under the point marked  $X$  at the surface of the soil.

(Civil Services Exam. 1999)

**Solution :** Divide the area into three rectangles  $ABDC$ ,  $CDFE$  and  $DGHF$  so that point  $X$  falls at a the corner of each one of these.

$$\begin{aligned} \text{Now } S_i &= \frac{q \cdot B}{E_s} (1 - \mu^2) I_w \\ &= \frac{q}{E_s} (1 - \mu^2) (B_1 I_{w1} + B_2 I_{w2} + B_3 I_{w3}) \end{aligned}$$

where  $I_{w1} = 0.77$  for  $L/B = 80/40 = 2$

of first rectangle ;  $B_1 = 40 \text{ m}$

$I_{w2} = 0.56$  for  $L/B = 40/40 = 1$

of second rectangle ;  $B_2 = 40 \text{ m}$

and  $I_{w3} = 0.77$  for  $L/B = 40/20 = 2$

of third rectangle ;  $B_3 = 20 \text{ m}$

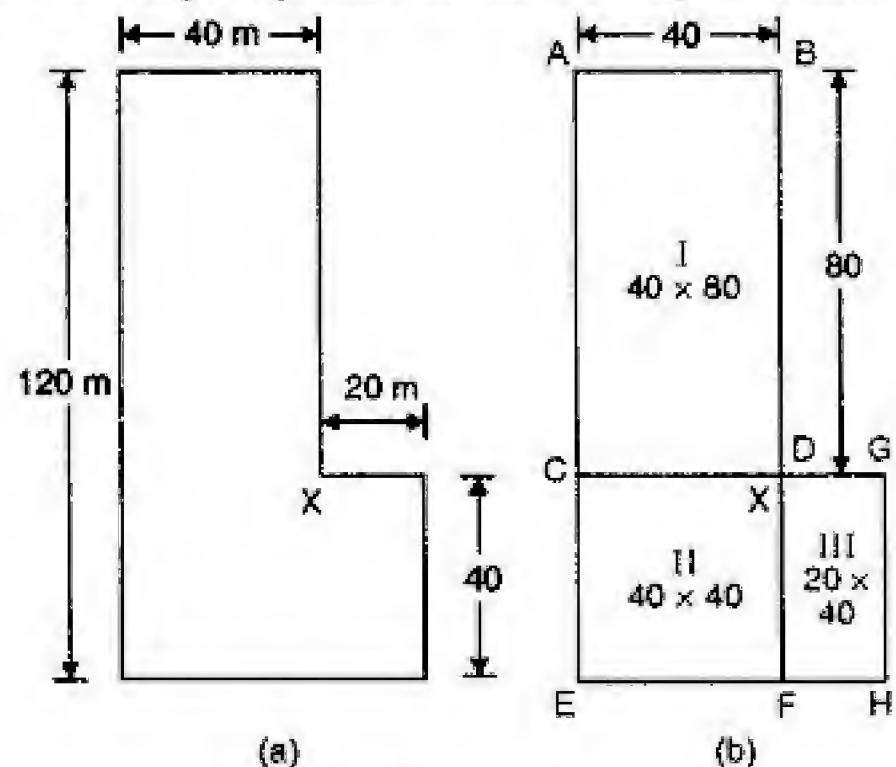


FIG. 25.14

$$q = 150 \text{ kN/m}^2 ; E = 15 \text{ MN/m}^2 = 15000 \text{ kN/m}^2 ; \mu = 0.5 \text{ (assumed for soft clay).}$$

$$\therefore S_i = \frac{150}{15000} (1 - 0.5)^2 [40 \times 0.77 + 40 \times 0.56 + 20 \times 0.77] = 0.5145 \text{ m} = 51.45 \text{ cm}$$

**Example 25.9.** Proportion a strap footing for the following data :

Allowable pressures :  $150 \text{ kN/m}^2$  for DL + reduced LL

$225 \text{ kN/m}^2$  for DL + LL.

Column loads

	Column A	Column B
DL	500 kN	600 kN
LL	450 kN	800 kN.

Proportion the footing for uniform pressure under DL + reduced LL. Distance of C/C of column = 5.4 m; projection beyond column A not to exceed 0.5 m. (Civil Services Exam. 2001)

**Solution:** Refer Fig. 25.9.

Column A : D.L. + Reduced L.L. =  $500 + 0.5 \times 450 = 725 \text{ kN} = P_1$

Column B : D.L. + Reduced L.L. =  $600 + 0.5 \times 800 = 1000 \text{ kN} = P_2$

Basic equations to be used are :

$$R_1 = \frac{P_1 L}{L'} \quad \dots(1);$$

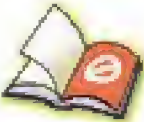
$$R_1 + R_2 = P_1 + P_2 \quad \dots(2)$$

$$L_s = 2 \left( e + \frac{C_1}{2} \right) \quad \dots(3) \quad \text{and}$$

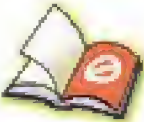
$$B = \frac{R_1}{L_s q_s}$$



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ally taken out and sand is filled in its place thus forming a 'sand pile'.

*Tension or uplift piles* anchor down the structures subjected to uplift due to hydrostatic pressure or due to overturning moment.

*Anchor piles* provide anchorage against horizontal pull from sheet piling or other pulling forces.

*Fender piles and dolphins* are used to protect water front structures against the impact from ships or other floating objects. *Sheet piles* are commonly used as bulkheads, or as impervious cutoff to reduce seepage and uplift under hydraulic structures. The *batter piles* are used to resist large horizontal or inclined forces.

**(b) Classification based on materials and composition.**

**1. Concrete piles :**

(a) Precast

(b) Cast-in-situ

(i) Driven piles : Cased or uncased

(ii) Bored piles : Pressure piles and under-reamed piles.

**2. Timber piles**

**3. Steel piles :**

(a) H-piles                      (b) Pipe pile                      (c) Sheet pile.

**4. Composite piles :**

(a) Concrete and timber

(b) Concrete and steel.

The *precast concrete piles* are generally used for a maximum design load of about 80 tonnes, except for large pre-stressed piles. They must be reinforced to withstand handling stresses. They require space for casting and storage, more time to set and cure before installation and heavy equipment for handling and driving. They also incur large cost in cutting off extra length or adding more length. The *cast-in-situ* piles are generally used for a maximum design load of 75 tonnes, except for compacted, pedestal piles. They are installed by pre-excavation, thus eliminating vibration due to driving and the handling stresses. Cast-in-place piles may be classified into two classes : driven piles (cased or uncased) and bored piles (pressure piles, pedestal piles and under-reamed piles). A variety of cast-in-situ piles are in use, each bearing the name of the manufacturer. The common types are as follows :

(i) Raymond standard pile,

(ii) Raymond step-taper pile,

(iii) Union metal pile of monotube,

(iv) MacArthur compressed uncased pile,

(v) MacArthur cased pile,

(vi) Franki standard pile,

(vii) Western button bottom pile etc.

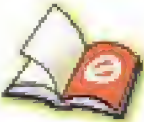
*Under-reamed pile* is special type of bored pile having an increased diameter or bulb at some point in its length, to anchor the foundation in expansive soil subjected to alternate expansion and contraction.

*Concrete filled steel piles* and *steel H-pile* are used as long piles with high bearing capacity. They are rarely used unless they reach a stratum of exceptionally high supporting capacity, since their cost is very high. *Timber piles* have small bearing capacity, and are not permanent unless treated. They are prone to damage by hard driving, and should not



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4. For clays, the dynamic formulae are valueless because the skin friction developed in clay during driving is very much less (due to change in soil structure from flocculant to dispersed or due to thixotropic effect) than which occurs after a period of time. Also, the point resistance is much more at the time of driving because of pore pressure developed in clay, which reduces later on when the pore pressure dissipate.

5. Dynamic formulae give no indication about probable future settlement or temporary changes in soil structure.

6. The formulae do not take into account the reduced bearing capacity of pile when in a group.

7. Law of impact used for determining energy loss is not strictly valid for piles subjected to restraining influence of the surrounding soil.

8. In Engineering News formula, the weight of the pile and hence its inertia effect is neglected.

9. Energy losses due to vibrations, heat and damage to dolly or packing are not accounted for.

10. In Hiley's formula, a number of constants are involved, which are difficult to determine.

## 26.5. STATIC FORMULAE

The static formulae are based on assumption that the ultimate bearing capacity  $Q_{up}$  of a pile is the sum of the total ultimate skin friction  $R_f$  and total ultimate point or end bearing resistance  $R_p$  :

$$Q_{up} = R_f + R_p \quad \dots(26.8)$$

$$\text{or} \quad Q_{up} = A_s \cdot r_f + A_p \cdot r_p \quad \dots(26.9)$$

where  $A_s$  = surface area of pile upon which the skin friction acts

$A_p$  = area of cross-section of pile on which bearing resistance acts. For tapered piles,  $A$  may be taken as the crosssectional area at the lower one-third of the embedded length.

$r_f$  = average skin friction

$r_p$  = unit point or toe resistance

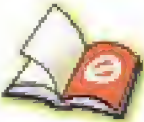
A factor of safety of 2.5 or 3 may be adopted for finding the allowable load.

(i) **For cohesive soil.** For the pile in cohesive soil, point bearing is generally neglected for individual pile action, since it is negligible as compared to frictional resistance. The unit skin friction may be taken equal to the shear strength of the soil multiplied by a reduction factor  $\alpha$  (or  $m$ ) :

$$\begin{aligned} \text{Thus} \quad r_f &= \text{average skin friction along the length of the pile} \\ &= \alpha \bar{c} \text{ (or } m \bar{c} \text{)}, \end{aligned} \quad \dots(26.10)$$

$$\text{and} \quad r_p = c_p N_c = 9 c_p \quad \dots(26.11)$$

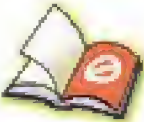
$$\therefore Q_{up} = m \bar{c} A_s + 9 c_p A_p \quad \dots(26.12)$$



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**Example 26.1.** A wooden pile is being driven with a drop hammer weighing 20 kN and having a free fall of 1.0 m. The penetration in the last blow is 5 mm. Determine the load carrying capacity of the pile according to the Engineering News Formula.

**Solution.** 
$$Q_a = \frac{WH}{6(S + C)}$$

where  $W = 20 \text{ kN}$  ;  $H = 1 \times 100 = 100 \text{ cm}$  ;  $S = 0.5 \text{ cm}$  ;  $C = 2.5 \text{ cm}$

$$Q_a = \frac{20 \times 100}{6(0.5 + 2.5)} = 111.1 \text{ kN.}$$

**Example 26.2.** In a 16 pile group, the pile diameter is 45 cm and centre to centre spacing of the square group is 1.5 m. If  $c = 50 \text{ kN/m}^2$ , determine whether the failure would occur with the pile acting individually, or as a group? Neglect bearing at the tip of the pile. All piles are 10 m long. Take  $m = 0.7$  for shear mobilisation around each pile

**Solution**  $n = 16$  ;  $d = 45 \text{ cm}$  ;  $L = 10 \text{ m}$

$$\text{Width of group} = B = (150 \times 3) + 45 = 495 \text{ cm} = 4.95 \text{ m}$$

(i) For the group

$$Q_{ug} = c \times \text{perimeter} \times \text{length} = c \times 4B \times L = 50 \times 4 \times 4.95 \times 10 = 9900 \text{ kN.} \quad \dots(i)$$

(ii) For the piles acting individually :

$$Q_{ug} = n Q_{up} = n \{ mcA_p \}$$

$$A_p = \pi dL = \pi \times 0.45 \times 10$$

$$\therefore Q_{ug} = 16 \times 0.7 \times 50 \times \pi \times 0.45 \times 10 = 7917 \text{ kN} \quad \dots(ii)$$

which is less than the load carried by the group action. Hence the foundation will fail by the piles acting individually, and the load at failure would be 7917 kN.

**Example 26.3.** A  $n$ -pile group has to be proportioned in a uniform pattern in soft clay with equal spacing in all directions. Assuming any value of  $c$ , determine the optimum value of spacing of piles in the group. Take  $n = 25$  and  $m = 0.7$ . Neglect the end bearing effect and assume that each pile is circular in section.

**Solution :** The optimum spacing of individual piles is based on the premise that the efficiency of the pile group is unity. That is, the total load carried by the piles, acting individually is equal to the load carried by group action.

Let the centre to centre spacing of each pile =  $s$ . Diameter of each pile =  $d$

$$n = 25$$

$\therefore$  Width for each pile group  $4s + d$  ; Length of pile =  $L$

$\therefore$  Load carried by group action is

$$Q_{ug1} = c \{ 4(4s + d) L \} = 4cL(4s + d) \quad \dots(i)$$

Load carried by pile acting individually is

$$Q_{ug2} = n(mc)(\pi dL) = 25 \times 0.7 c \pi dL = 55dL \cdot c \quad \dots(ii)$$

Equating (i) and (ii) for the optimum spacing



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$$\frac{d^4 y}{dx^4} + \frac{E_s}{EI} y = 0 \quad \dots(26.43)$$

The applied lateral load  $P_l$  and an applied moment  $M_l$  are considered separately. According to the principle of superposition, the above equation becomes

$$\text{For case } A \quad \frac{d^4 y_A}{dx^4} + \frac{E_s}{EI} y_A = 0 \quad \dots(26.44)$$

$$\text{For case } B \quad \frac{d^4 y_B}{dx^4} + \frac{E_s}{EI} y_B = 0 \quad \dots(26.45)$$

Substituting the definitions of non-dimensional parameters, a non-dimensional differential equation can be written for case  $A$  as

$$\frac{d^4 A_y}{dZ^4} + \phi(Z) A_y = 0 \quad \dots(26.46)$$

$$\text{and for case } B, \quad \frac{d^4 B_y}{dZ^4} + \phi(Z) B_y = 0 \quad \dots(26.47)$$

The solution of these two given equations give the values of coefficients  $A_y$  and  $B_y$ . Similarly, by successive integration of these two equations, one can get the expressions, for slope, moment, shear and soil reaction and their respective coefficients ( $A_s, B_s$ ), ( $A_m, B_m$ ), ( $A_v, B_v$ ) and ( $A_p, B_p$ ) can be analytically computed. Table 26.2 gives the analytical values of these coefficients, computed by Reese and Matlock (1956).

TABLE 26.2. NON-DIMENSIONAL COEFFICIENTS FOR Laterally LOADED PILE  
(REESE AND MATLOCK)

$Z$	$A_y$	$A_s$	$A_m$	$A_v$	$A_p$
0.0	2.435	- 1.623	0.000	1.000	0.000
0.1	2.273	- 1.618	0.100	0.989	- 0.227
0.2	2.112	- 1.603	0.198	0.956	- 0.422
0.3	1.952	- 1.578	0.291	0.906	- 0.586
0.4	1.796	- 1.545	0.379	0.840	- 0.718
0.5	1.644	- 1.503	0.459	0.764	- 0.822
0.6	1.496	- 1.454	0.532	0.677	- 0.897
0.7	1.353	- 1.397	0.595	0.585	- 0.947
0.8	1.216	- 1.335	0.649	0.489	- 0.973
0.9	1.086	- 1.268	0.693	0.392	- 0.977
1.0	0.962	- 1.197	0.727	0.295	- 0.962
1.2	0.738	- 1.047	0.767	0.109	- 0.885
1.4	0.544	- 0.893	0.772	- 0.056	- 0.761
1.6	0.381	- 0.741	0.746	- 0.193	- 0.609
1.8	0.247	- 0.596	0.696	- 0.298	- 0.445
2.0	0.142	- 0.464	0.628	- 0.371	- 0.283
3.0	- 0.075	- 0.040	0.225	- 0.349	0.226
4.0	- 0.050	0.052	0.000	- 0.106	0.201
5.0	- 0.009	0.025	- 0.033	0.013	0.046



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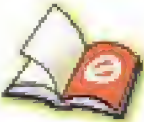


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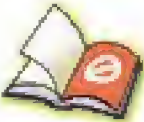


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$$r_p = c_u N_c = 9 c_u = 9 \times 40 = 360 \text{ kN/m}^2$$

$$A_{sg} = 4 B L_1 + 4 B L_2$$

$$r_{sg} = c_{u1} \text{ for first 8 m and } c_{u2} \text{ for next 10 m}$$

$$\therefore Q_{ug} = 12.25 \times 360 + 4 \times 3.5 [8 \times 25 + 10 \times 40] = 4410 + 8400 = 12810 \text{ kN} \quad \dots (2)$$

Lesser of (i) and (ii) = 12810 kN

$$\therefore \text{Safe load} = \frac{12810}{2.5} = 5124 \text{ kN}$$

**Example 26.23.** A group of 16 piles (diameter = 500 cm, length = 14 m, centre to centre spacing = 1 m) arranged in a square pattern passes through a recent fill (thickness = 3 m) overlying a soft clay deposit (thickness = 5 m) which is consolidating under the fill load and rests in a stiff clay strata (Fig. 26.37 a) All the strata are saturated. The soil properties of different strata are :

Type of soil	Unit weight ( $\gamma$ ) kN/m <sup>3</sup>	Strength parameters		Adhesion parameter ( $\alpha$ )
		$c_u$ (kPa)	$\phi_u$	
Fill	16	50	0	0.60
Soft clay	17	20	0	0.40
Stiff clay	21	70	0	0.45

Estimate the ultimate load capacity ( $Q_u$ ) of the pile group.

(Gate Exam. 2002)

**Solution :**

(a) Pile acting individually

$$Q_{un} = n (A_p r_p + A_s r_f)$$

where  $A_p = \frac{\pi}{4} (0.5)^2 = 0.1963 \text{ m}^2$  ;  $r_p = cN_c = 9 c_u = 9 \times 70 = 630 \text{ kN/m}^2$

$$A_s = \pi d (L_1 + L_2 + L_3) ; r_f = \alpha C_u \text{ for various strata.}$$

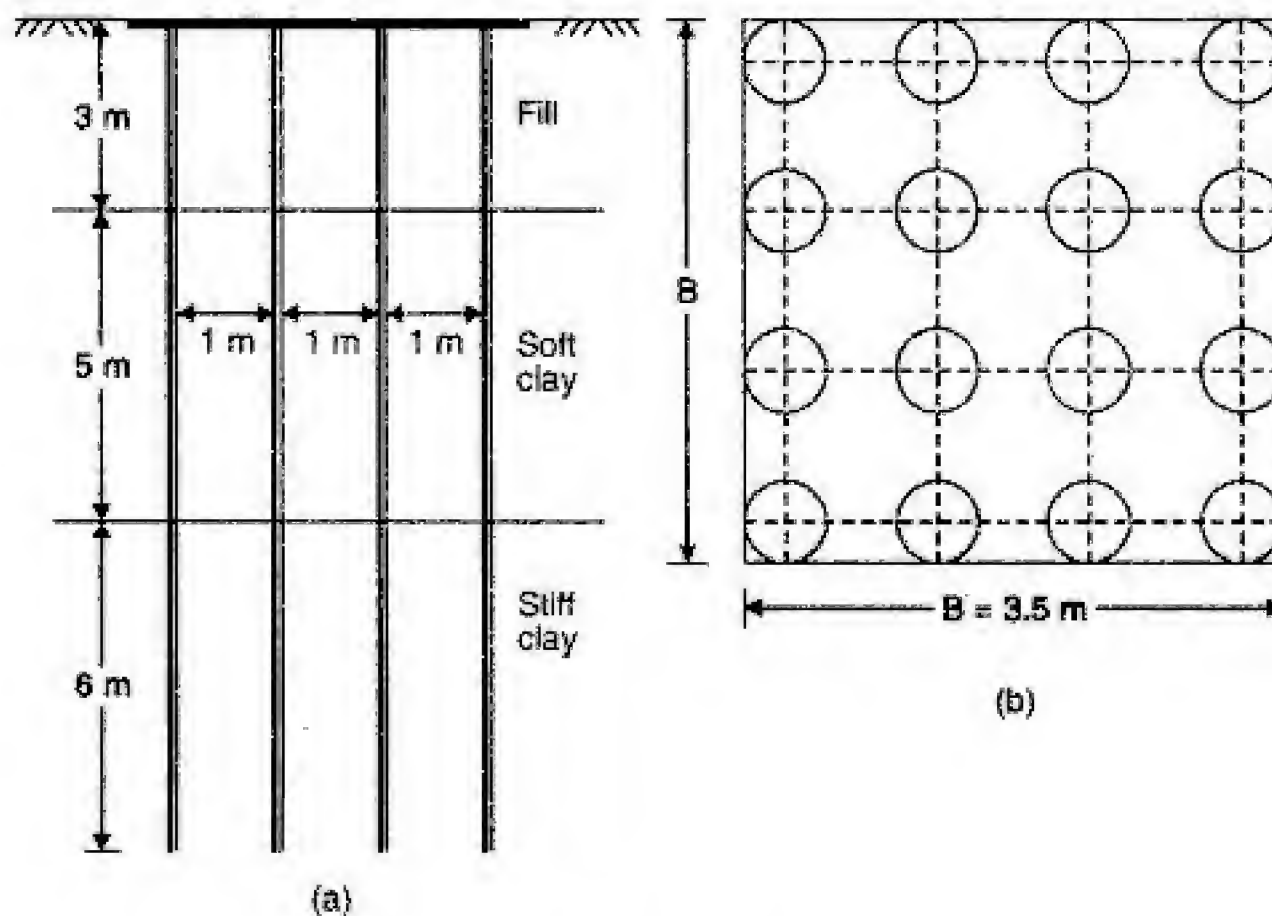


FIG. 26.37



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use of *phawrah Jhams* is effective. When power winches are available clayey strata can also be successfully excavated with the help of big grabs having tempered steel teeth.

As the well sinks deeper, the skin friction on the sides progressively increases. To overcome the increased skin friction and the loss in weight of the well due to buoyancy, additional loading known as *kentledge* is applied on the well.

Pumping out the water from inside the well is effective in sinking of well under certain conditions. Pumping should be discouraged in the initial stage. Unless the well has gone deep enough or has passed through a ring of clayey strata so that chances of tilts and shifts are minimised during this process. Complete dewatering should not be allowed when the well has been sunk to about 10 m depth. Sinking thereafter should be done by grabbing, chiselling, applying kentledge and using gelignite charges. Only when these methods have failed, dewatering may be allowed upto depressed water level of 5 m and *not more*.

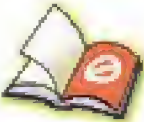
On certain occasions a well is struck up and normal method of kentledge and dredging fail to sink it further. In such a case frictional resistance developed on its outer periphery is reduced considerably by forcing jet of water on the outer face of the well around. This method is effective in case the well is being sunk in sand strata.

**4. Tilts and Shifts.** The primary aim in well sinking is to sink them straight and at the correct position. Suitable precautions should be taken to avoid tilts and shifts. Also proper records of tilts and shifts should be maintained and measure should be taken to counter-act tilts and shifts. The precautions to avoid tilts and shifts are as follows:

1. The outer surface of the well curb and steining should be as regular and smooth as possible.
2. The radius of curb should be kept 2 to 4 cm larger than the outside radius of well steining.
3. The cutting edge of the curb should be of uniform thickness and sharpness since the sharper edge has a greater tendency of sinking than a blunt edge.
4. The dredging should be done uniformly on all sides in a circular well and in both pockets of a twin well. The tilts and shifts of well, if any, must be carefully checked and recorded. The correct measurement of the tilts at any stage is perhaps one of the most important field observations required during well sinking.

As soon as tilt exceeds 1 in 200, the sinking should be supervised with special care and rectifying measures should be immediately taken. Any of the following measures can usefully be employed to counteract the tilts in the well during sinking operations :

**(i) Regulation of grabbing.** Unequal dredging causes tilts and hence if higher side is grabbed more by regulating the dredging, the tilt can be rectified [Fig. 27.12. (a)]. This method is not very effective when the well has been sunk to a great depth. In that case, a hole in the steining of the well is made on the higher side, and by hooks, the rope of the grab is pulled towards higher side to the maximum possible extent [Fig. 27.12 (b)]. The hole is made near the ground level. The well may be dewatered if possible and open excavation on the higher side is carried out.



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Fig. 28.2 (a) shows a simple spring with a spring constant  $k$  kg/m. When a weight  $W$  is attached to it (without any vibrations), it extends by an amount  $\delta_s$  [Fig. 28.2 (b)]. The static deflection  $\delta_s$  of the spring is given by

$$\delta_s = \frac{W}{k} \quad \dots(28.1)$$

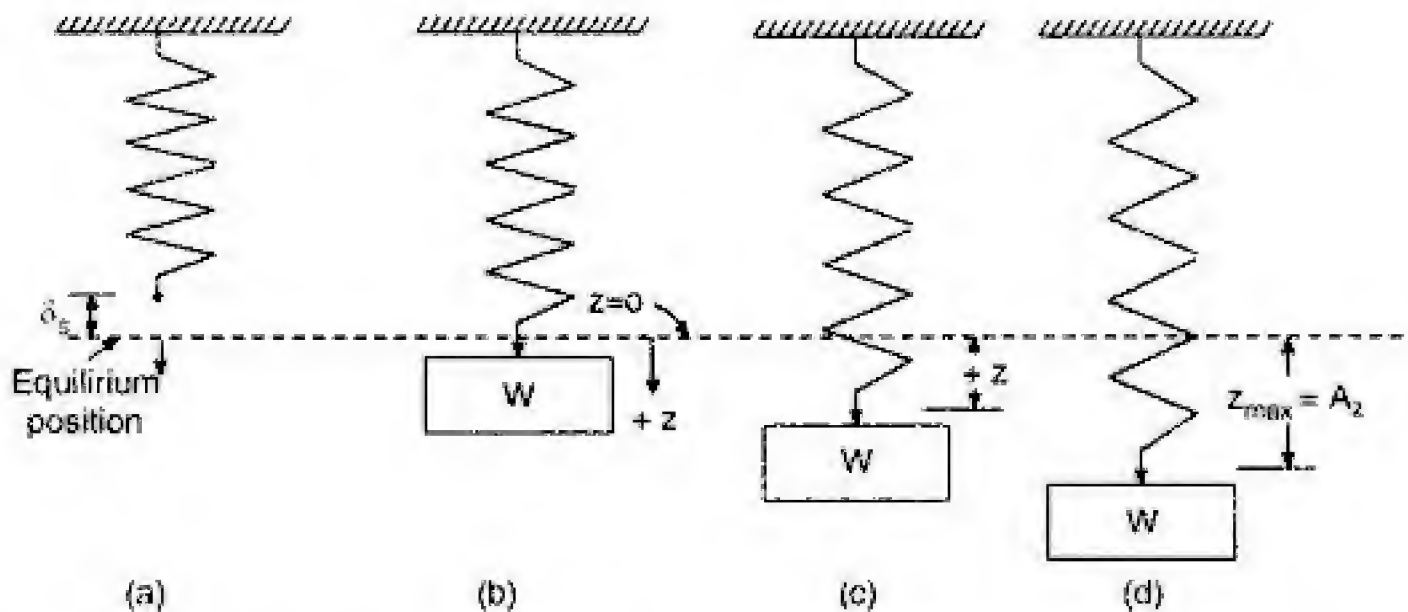


FIG. 28.2. FREE VIBRATIONS.

If the the spring mass system is pulled down, by an external force, by a maximum distance  $z_{max}$  or  $A_z$  (called the amplitude), and then released, the whole system vibrates with a certain frequency. Let  $z$  be the displacement of the mass at any instant, with respect to the equilibrium position, the force  $F_s$  in the spring ( $\uparrow$ ) is then given by

$$F_s = k (\delta_s + z) = k\delta_s + kz = W + kz \quad \dots(28.2)$$

The force acts in the opposite direction to the motion at any instant. The gravity force  $W$  acts downward. Hence when the motion is downward, the net downward force is equal to  $W \downarrow - (W + kz) \uparrow$ . This must be equal to mass  $\times$  acceleration. Hence, we get

$$W - (W + kz) = \frac{W}{g} \frac{d^2z}{dt^2}$$

$$\text{or} \quad \frac{W}{g} \frac{d^2z}{dt^2} + kz = 0 \quad \dots(28.3 \ a)$$

$$\text{which is usually written as} \quad m\ddot{z} + kz = 0 \quad \dots(28.3)$$

where  $m = \text{mass of the vibrating body} = W/g$   
 $\ddot{z} = \text{acceleration}$

Eq. 28.3 is called the *equation of motion*, which is similar to the following standard equation of motion :

$$\ddot{z} + \omega_n^2 z = 0 \quad \dots(28.4)$$

where  $\omega_n = \text{natural frequency of the system.}$

Comparing Eqs. 28.3 and 28.4,

$$\omega_n^2 = \frac{k}{m}$$



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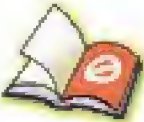
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TABLE 28.2. VALUE OF  $E_0$  AND  $\beta$ 

Soil type	$E_0$ (kg/cm <sup>2</sup> )	$\beta$ (kg/cm <sup>2</sup> /m)	$\beta$ (kg/cm <sup>2</sup> /m) ( $\times 10^6$ )
Dense sand and gravel	0	700 — 1150	7 — 11.5
Dense sand	0	400 — 58	4 — 5.8
Loose sand	0	70 — 140	0.7 — 1.4
Loose silty sand	7	58 — 115	0.58 — 1.15
Dense silty sand	21	230 — 460	2.30 — 4.60
Clay, semi-solid	56 — 105	11.5 — 23	0.115 — 0.23
Clay, stiff plastic	28 — 56	11.5 — 23	0.115 — 0.23
Clay, weak plastic	14 — 28	11.5 — 23	0.115 — 0.23

The spring constant for the truncated pyramid is calculated by first of all determining the surface deformation  $\delta_0$  given by following infinite integral :

$$\delta_0 = \frac{P_0}{\beta} \int_0^\infty \frac{dz}{(a + \alpha z)(b + \alpha z)(h + z)} \quad \dots(28.44 \ a)$$

or 
$$\delta_0 = \frac{P_0}{\beta b^2} \int_0^\infty \frac{dm}{(l + m)(1 + m)(s + m)} \quad \dots(28.44 \ b)$$

where  $l = \frac{a}{b}$  ( $a \geq b$ ) ;  $s = \frac{\alpha h}{b}$  ; and  $m = \frac{\alpha z}{b}$  ...(28.45)

The equivalent soil spring constant  $k$  in the vertical plane is given by

$$k = \frac{P_0}{\delta_0} \text{ (by definition)}$$

$$\frac{1}{k} = \frac{1}{\beta b^2} \int_0^\infty \frac{dm}{(l+m)(1+m)(s+m)} \quad \dots(28.46)$$

If the base of foundation is circular, a truncated cone will be considered in the place of truncated prism, and the above expression will be modified as under :

$$\frac{1}{k} = \frac{1}{\beta b^2} \int_0^\infty \frac{dm}{(1+m)^2(s+m)} \quad \dots(28.47)$$

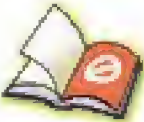
where  $m = \frac{\alpha z}{b}$  and  $s = \frac{\alpha h}{b}$  ;  $b$  = diameter of the foundation.

The values of  $k$  are determined by curves of Fig. 28.7 in which  $k$  is given by the equation

$$k = \beta b^2 \lambda \text{ (for rectangular base)} \quad \dots(28.48)$$

and  $k = \frac{\pi}{4} \beta b^2 \lambda$  (for circular base of dia.  $b$ ). ...(28.48 a)

For given values of  $\frac{\alpha h}{b}$  and  $\frac{a}{b}$  ratios,  $\frac{\lambda b}{a}$  is determined from the curves, and thus  $\lambda$  is known. Then  $k$  is calculated from Eq. 28.38.



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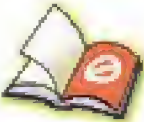
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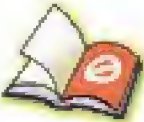
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## Design of Flexible Pavement

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### 29.1. INTRODUCTION : TYPES OF PAVEMENTS

A natural earth track is incapable of supporting modern wheel loads. A constructed pavement is required on the top of the soil in order to distribute the wheel load efficiently and to provide the necessary wearing surface. *A pavement is, therefore, defined as a relatively stable crust constructed over the natural soil for the purpose of supporting and distributing the wheel loads and providing an adequate wearing surface.* Depending upon the mode of supporting and distributing loads, pavements are classified as flexible, rigid and semi-flexible.

The *flexible pavements* consist of a relatively thin wearing surface built over a base course and sub-base course, and they rest on compacted sub-grade. The flexible pavements are able to resist only very small tensile stresses. In contrast to this, *rigid pavements* are made up of Portland cement concrete and may or may not have a base course between the pavement and the sub-grade. A rigid pavement can take appreciable tensile stresses and is capable of bridging small weakness and depression in the sub-grade. A *semi-flexible* pavement is made of dry-lean concrete or soil cement, and possesses qualities intermediate between the flexible and rigid pavements. A semi-flexible pavement possesses appreciable flexural strength but its modulus of elasticity is considerably lower than that of concrete.

The essential difference between rigid and flexible pavements is the manner in which they distribute the load over the sub-grade. The design of a flexible pavement is based on the principle that a surface load is dissipated by carrying it deep into the ground through successive layer of granular materials. Hence the strength of a flexible layer is a result of building up thick layers and thereby distributing the load over the sub-grade rather than by the bending action. The thickness design of the flexible pavement is influenced primarily by the strength of the sub-grade.

Because of its rigidity and high tensile strength, a rigid pavement tends to distribute the load over a relatively wide area of soil, and a major portion of the structural capacity is supplied by the slab itself. For this reason, minor variations in sub-grade strength have little influence upon the structural capacity of the pavement.

The rigid pavements are used for heavier loads and can be constructed over relatively poor sub-grade such as black cotton or plastic soils, peat etc. However, since the load is taken up by the bending action of the slab, uniform sub-grade support is the most



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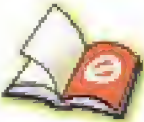
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$h$  = slab thickness (cm)

$\mu$  = Poisson's ratio for concrete

$l$  = radius of relative stiffness (cm)

$E$  = modulus of elasticity ( $\text{kg}/\text{cm}^2$ )

$k_s$  = modulus of sub-grade reaction ( $\text{kg}/\text{cm}^3$ )

$b$  = radius of equivalent distribution of pressure (cm), given by the following equations :

$$b = \sqrt{1.6a^2 + h^2} - 0.675h, \text{ when } a < 1.724h \quad \dots(30.6)$$

$$b = a \text{ when } a \geq 1.724h \quad \dots(30.7)$$

$a$  = radius of area of contact between the slab (cm), the area being assumed circular in case of corner and interior loads, and semi circular for edge loads

$c_1, c_2$  = correlation factors to allow for a redistribution of sub-grade reactions ;

$$c_1 \approx 5l ; c_3 \approx 0.2.$$

Pickett (1951) gave the following semi-empirical formulae for corner loading for protected and unprotected corners :

For slab corners protected by load-transfer devices such as bars etc. :

$$(\sigma)_{\text{corner}} = \frac{3.36 P}{h^2} \left[ 1 - \frac{\sqrt{a/l}}{0.925 + 0.22 a/l} \right] \quad \dots(30.8)$$

For unprotected corners :

$$(\sigma)_{\text{corner}} = \frac{4.2 P}{h^2} \left[ 1 - \frac{\sqrt{a/l}}{0.925 + 0.22 a/l} \right] \quad \dots(30.9)$$

The above two formulae are in FPS units in which  $P$  in lbs,  $\sigma$  is in  $\text{lb}/\text{in}^2$ ,  $l$ ,  $a$  and  $h$  are in inches and  $k_s$  is in  $\text{lb}/\text{in}$ .

The formulae for the stresses in pavements are complicated for direct calculations. Use may be made of the influence diagrams and charts prepared by Docker (1948) and Pickett (1951).

### 30.4. STRESSES DUE TO WARPING

The surface of the slab is subjected to wide range of temperature during the daily cycle, whereas the temperature of the bottom of the slab in contact with the sub-grade of the base remains relatively more constant. This temperature gradient through the slab causes differential expansion or contraction between the top and bottom of the slab.

In the day, top surface expands more than the bottom and the slab assumes a shape convex upwards. The weight of the slab and load transfer devices or friction at joint will restrain free warping and will tend to bend it back into its original shape, and hence compressive stresses at top and tensile at the bottom are created.

In the night, the sides and corners warp upwards, and might actually leave the sub-grade. In this position, the weight of the raised portions of the slab tend to bend them down.



Hence tension at top and compression in the bottom is developed. Bradbury (1938) gave the following equations for edge stress and interior stress due to warping :

$$(\sigma)_{edge} = \frac{CE \epsilon_t \Delta t}{2} \quad \dots(30.10)$$

$$(\sigma)_{interior} = \frac{E \epsilon_t \Delta t}{2} \left( \frac{C_x + \mu C_y}{1 - \mu^2} \right) \quad \dots(30.11)$$

where  $\epsilon_t$  = co-efficient of expansion of concrete ( $5 \times 10^{-6}$  per  $^{\circ}\text{F}$ , average)

$\Delta t$  = total difference in temperature (may be taken to be equal to about  $1^{\circ}\text{F}$  per 1 cm thickness of slab)

$E$  = modulus of elasticity of concrete

$\mu$  = Poisson's ratio

$C$  = co-efficient given by Fig. 30.1, corresponding to  $L/l$  ratio

$C_x$  = co-efficient in the desired direction (say,  $x$  direction)

$C_y$  = co-efficient in the perpendicular direction (say,  $y$  direction)

$L$  = length of the edge

$L_x$  = free length in  $x$  direction (i.e., the direction in which  $\sigma_{interior}$  is sought)

$L_y$  = free width in  $y$  direction.

### 30.5. STRESSES DUE TO SUB-GRADE FRICTION

Stresses can also be set up in rigid pavements due to *uniform temperature change*, which cause the slab to contract or expand. If the slab is free to move and there is no friction between the slab and the sub-grade, no stresses will result. However, if friction exists between the slab and sub-grade, restraint results from the friction forces and the slab is stressed. During expansion, the under-side of the slab is subjected to compressive stress, while during contraction tensile stresses are induced due to the sub-grade friction. Fig. 30.2 (b) shows the distribution of frictional stresses, as suggested by Kelley (1939). Test results have shown that fully mobilised frictional resistance  $f_m$  is realised for a distance  $\frac{L}{2} - x$ , but from there to the centre of the slab, the stress distribution is parabolic in shape. The equations for the *average co-efficient of sub-grade resistance*  $f$  are as follows :

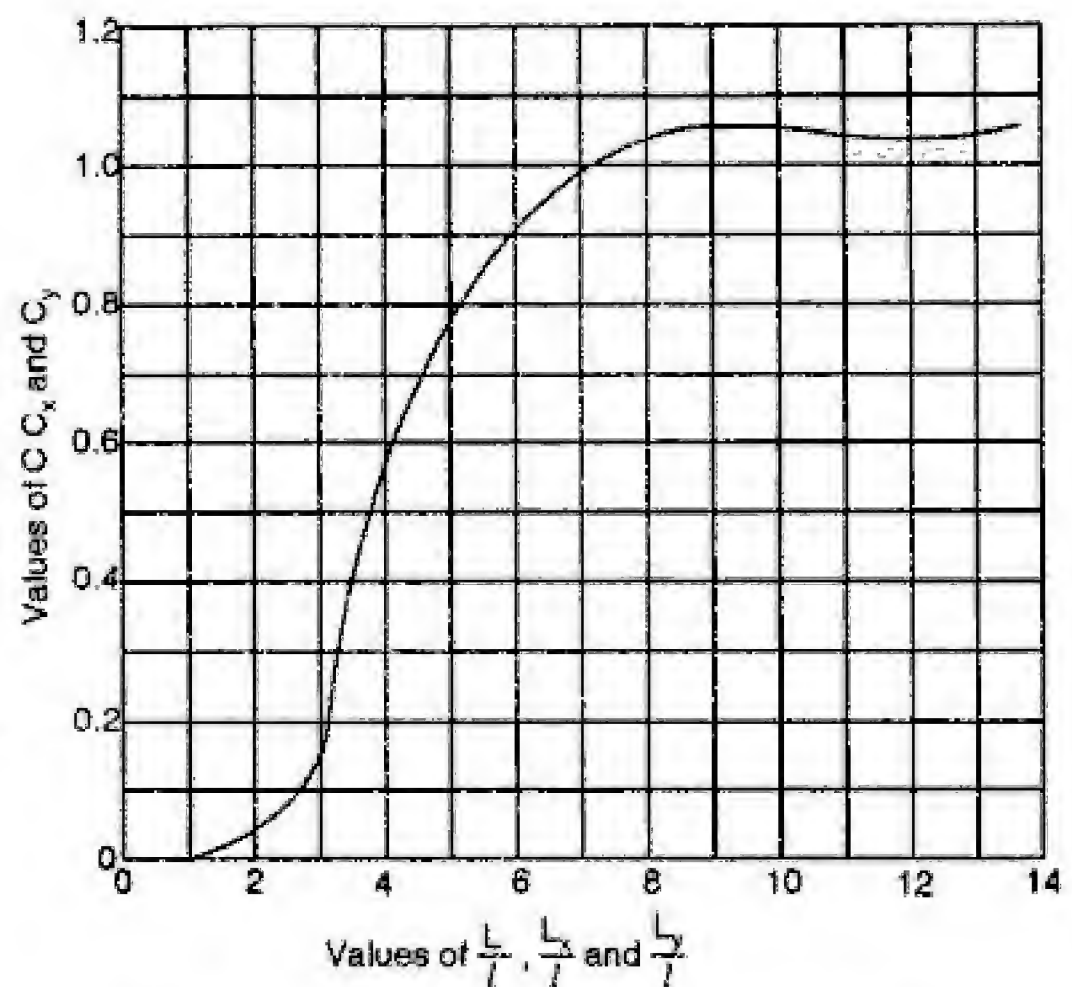


FIG. 30.1. WARPING STRESS CO-EFFICIENTS (BRADBURY, 1938).

$$\text{For } x < \frac{1}{2} L, f = f_m \left( 1 - \frac{2x}{3L} \right) \quad \dots(30.12)$$

$$\text{For } x > \frac{1}{2} L, f = \frac{2f_m}{3} \sqrt{\frac{L}{2x}} \quad \dots(30.13)$$

It has been shown that the minimum amount of displacement required for friction to be fully developed is 0.06 inch (1.5 mm).

For equilibrium conditions, the summation of the friction forces from the centres of the slab to the free end must be equal to the total tension in the concrete. The frictional resistance is assumed to be a finite value, dependent upon the weight of the slab and coefficient of sliding friction.

Let  $W$  = weight of the slab per sq. metre ( $\text{kg/m}^2$ )

$L$  = length of the slab, in metres

$h$  = thickness of the slab in cm

$\sigma_0$  = stress in concrete,  $\text{kg/cm}^2$

$f$  = average co-efficient of sub-grade resistance ( $f = 2.3$  to  $1.15$  ; average  $1.5$ )

Then frictional resistance up to the centre of the slab

$$= W \times \frac{L}{2} \times 1 \times f \text{ (kg) (per metre width of the slab)} \quad \dots(1)$$

$$\text{Total tension in the concrete} = (1 \times 100)h \sigma_0 \text{ (per metre width of the slab)} \quad \dots(2)$$

$$\text{Equating (1) and (2), we get } \frac{WLf}{2} = 100 h \sigma_0$$

$$\text{and } \sigma_0 = \frac{WLf}{200h} \text{ kg/cm}^2 \quad \dots(30.14)$$

Taking unit weight of concrete as  $2400 \text{ kg/m}^3$ ,

$$W = 1 \times 1 \times \frac{h}{100} \times 2400 = 24h \text{ kg} \quad \dots(30.15)$$

Substituting in equation 30.14, we get

$$\sigma_0 = \frac{3Lf}{25h} \quad \dots(30.16)$$

More exact calculations could be made by taking into account the variation in friction between the free end and the centre of the slab.

**Combined stresses due to load and temperature :** The critical stress condition in the slab is the one in which stresses due to temperature and stresses due to load are additive. This condition can result when the slab corners are warped down ward and a load is applied at the interior of the slab. Similarly, during a clear, cool night followed

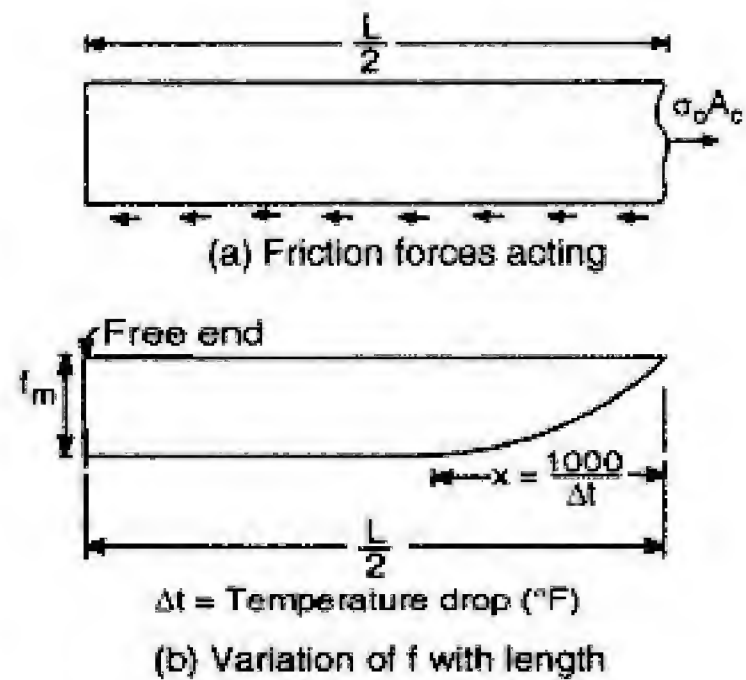


FIG. 30.2. STRESSES RESULTING FROM CONTRACTION.

by a hot, sunny day, the edges and corners of the slab will warp upward and the corners may then be only partially supported or even unsupported. This condition will give the maximum stresses due to loading.

### 30.6. DESIGN METHOD

Portland Cement Association (1951) has developed a design procedure for rigid highway pavements based upon formulae developed by Pickett (Eqs. 30.8 and 30.9). Fig. 30.3 and 30.4 show the design charts for protected and unprotected corners, based on the formulae by Pickett for the design of highway pavements. The values of  $k_s$  are in  $\text{lb/in}^3$ .

The arrows shown on the diagrams indicate the procedure for finding the pavement thickness  $h$  for a given value of allowable stress in concrete, magnitude of wheel load and given value of  $k$ , and also for finding the stress in the pavement of a given thickness corresponding to a given wheel load and a given value of  $k$ . The working stress in the concrete that is used in the analysis is obtained by dividing the modulus of rupture of the concrete by a safety factor of 2.0. Loads which are applied to the pavement are increased 20 per cent to account for impact. Concrete pavements designed with the help of these charts are considered to have excess strength to offset working stresses.

Fig. 30.5, 30.6 and 30.7 give the PCA (Portland Concrete Association) design charts for Rigid airport pavements, with various wheel loads and tyre pressure (indicated dotted lines).

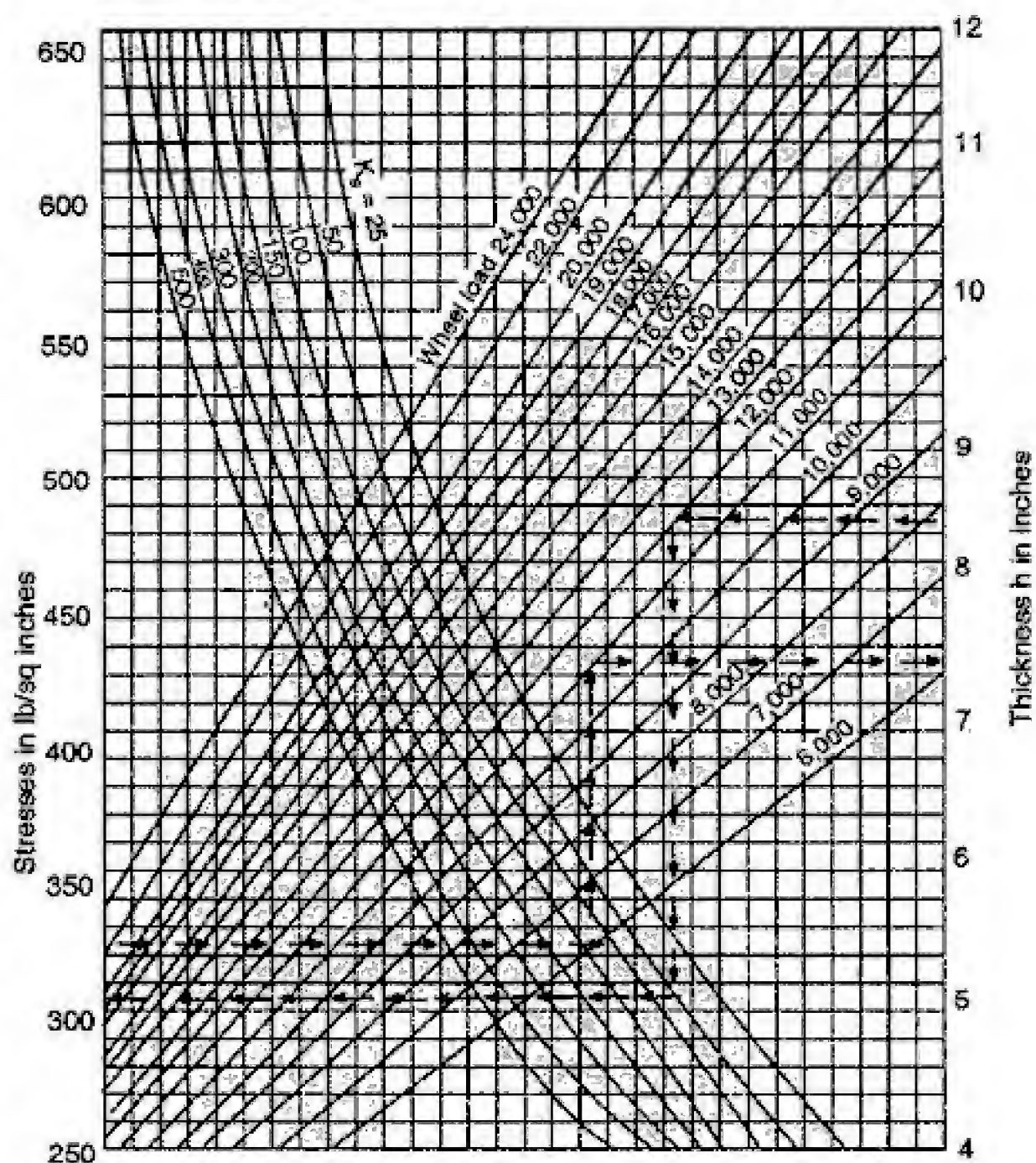


FIG. 30.3. PCA DESIGN CHARTS FOR RIGID HIGHWAY PAVEMENTS PROTECTED CORNERS, FOR LOADS ON DUAL TYRES.



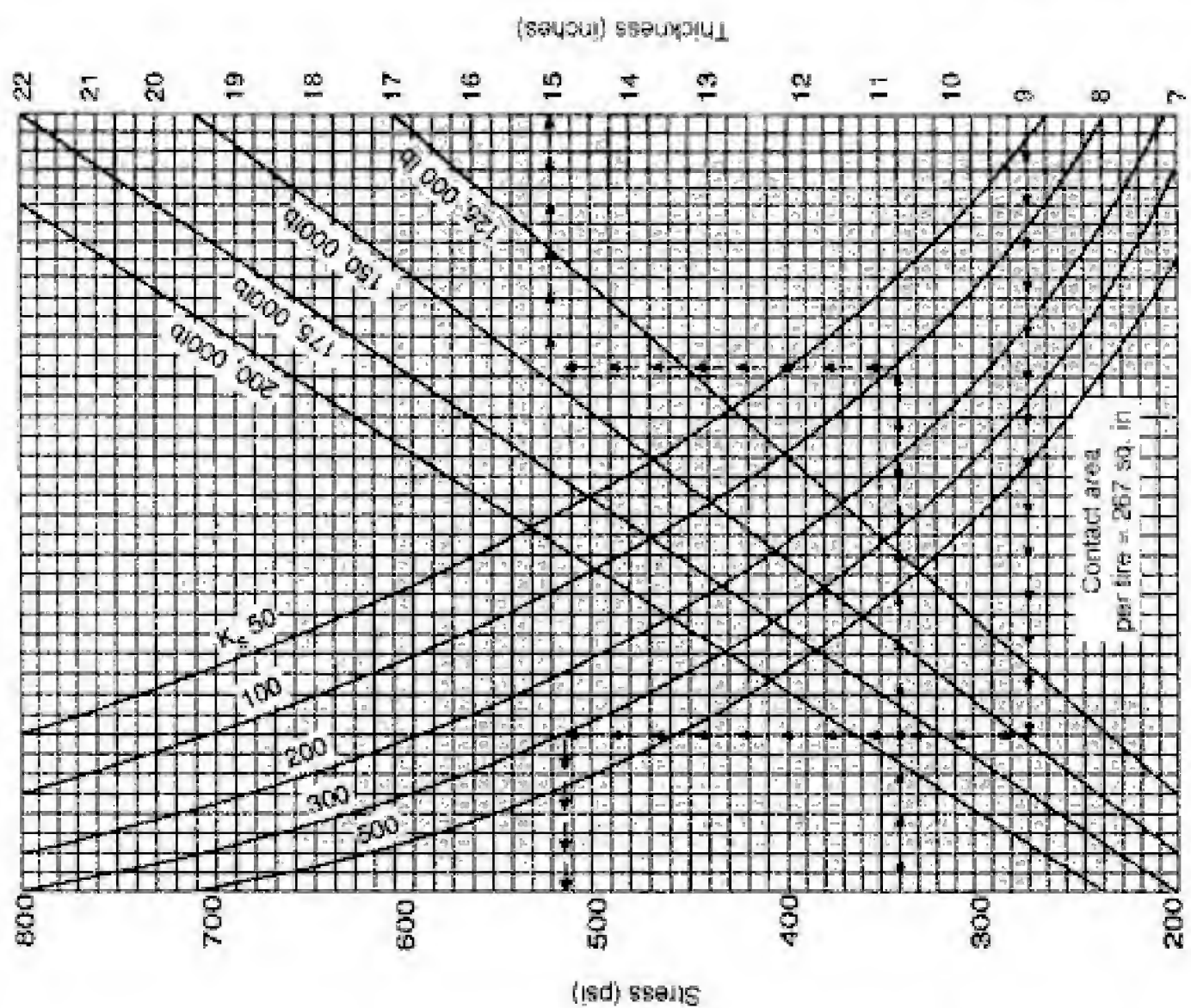


FIG. 30.6. PCA DESIGN CURVES FOR RIGID AIRPORT PAVEMENT, DUAL WHEELS: INTERIOR LOAD  
( $E = 4,000,000$ , psi ;  $\mu = 0.15$ ).

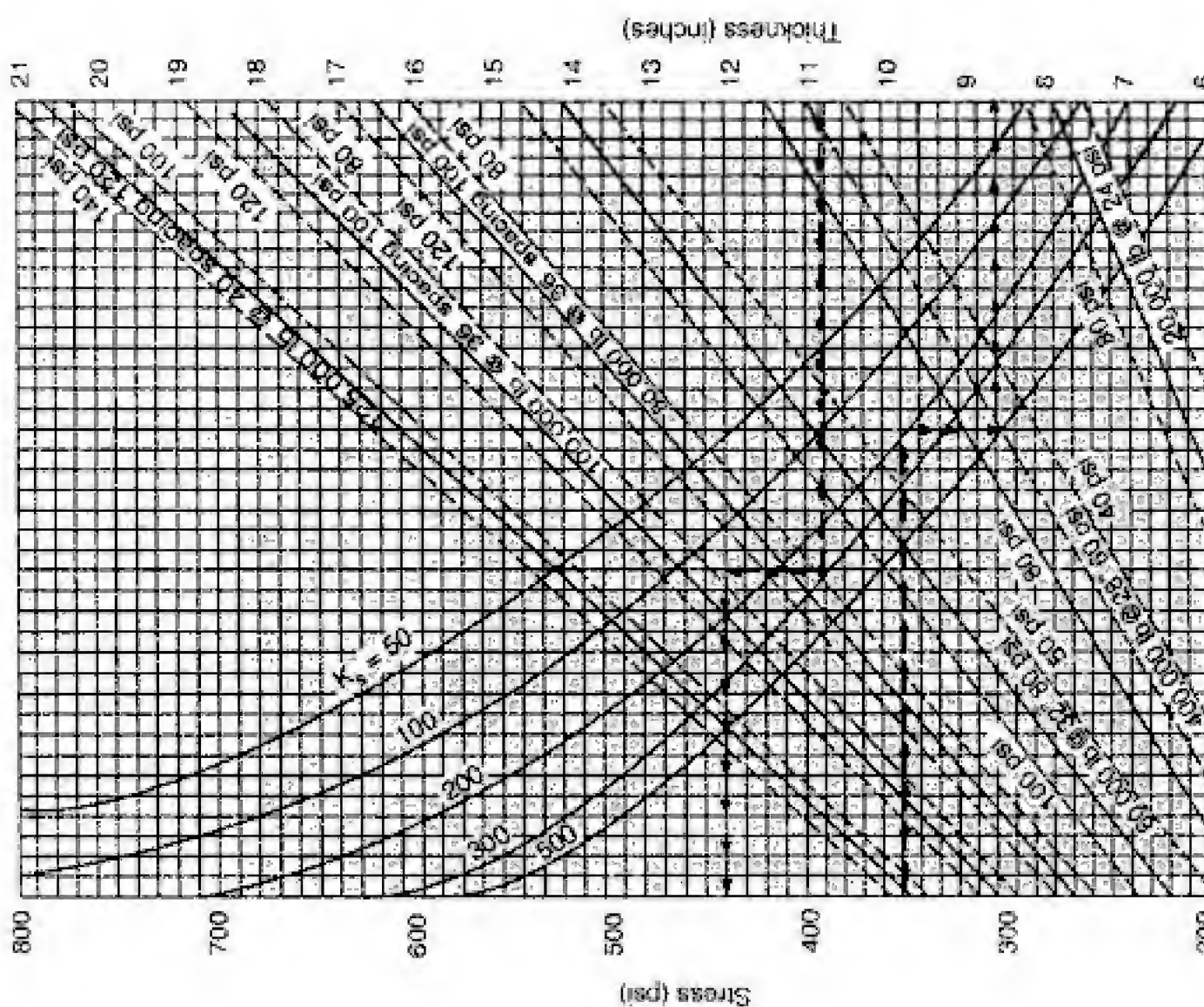


FIG. 30.7. PCA DESIGN CURVES FOR RIGID-AIRPORT PAVEMENT, DUALTANDEM WHEEL, INTERIOR LOAD  
( $E = 4,000,000$ , psi ;  $\mu = 0.15$ ).

## Stabilisation of Soils

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### 31.1. INTRODUCTION

*Stabilisation, in a broad sense, incorporates the various methods employed for modifying the properties of a soil to improve its engineering performance.* Stabilisation is being used for a variety of engineering works, the most common application being in the construction of road and air-field pavements, where the main objective is to increase the strength or stability of soil and to reduce the construction cost by making best use of the locally available materials. Methods of stabilisation may be grouped under two main types : (a) modification or improvement of a soil property of the existing soil without any admixture, and (b) modification of the properties with the help of admixtures. Compaction and drainage are the examples of the first type, which improve the inherent shear strength of soil. Examples of the second type are : mechanical stabilisation, stabilisation with cement, lime, bitumen and chemicals etc. Some of the more commonly used methods will be discussed in this chapter.

### 31.2. MECHANICAL STABILISATION

Mechanical stabilisation involves two operations : (i) changing the composition of soil by addition or removal of certain constituents, and (ii) densification or compaction. The particle size distribution and composition are the important factors governing the engineering behaviour of a soil. Significant changes in the properties can be made by addition or removal of suitable soil fractions. For mechanical stabilisation, where the primary purpose is to have a soil resistant to deformation and displacement under loads, soil materials can be divided into two fractions : the *granular fraction* retained on a 75 micron IS sieve and the *fine soil fraction* passing a 75-micron sieve. The granular fraction impart strength and hardness. The fine fraction provides cohesion or binding property, water-retention capacity and also acts as a filler for the voids of the coarse fraction.

Mechanical stabilisation has been largely used in the construction of cheap roads. Guide of specifications have been drawn for gradation requirements of the bases and surfacing. Typical examples are given in Table 31.1. Instead of strictly observing the specifications, emphasis should be laid on making the maximum use of the locally available materials as many materials are found to be quite satisfactory under load conditons.



**TABLE 31.1. TYPICAL GRADATION SPECIFICATIONS OF MECHANICALLY STABILISED BASES AND SURFACINGS**

<i>IS Sieve</i>	<i>Percentage passing</i>			
	<i>Base</i>		<i>Surfacing</i>	<i>Base or surfacing</i>
	<i>Max. size</i>		<i>Max. size</i>	<i>Max. size</i>
	<i>40 mm</i>	<i>20 mm</i>	<i>20 mm</i>	<i>10 mm</i>
40 mm	100	—	—	—
20 mm	80 – 100	100	100	—
10 mm	55 – 80	80 – 100	80 – 100	100
4.75 mm	40 – 60	50 – 75	60 – 85	80 – 100
2.0 mm	30 – 50	35 – 60	45 – 70	50 – 80
1.18 mm	—	—	35 – 60	40 – 65
600 micron	15 – 35	15 – 30	—	—
300 micron	—	—	20 – 40	20 – 40
75 micron	5 – 15	5 – 15	10 – 25	10 – 25

**Note.** (i) For bases : Liquid limit not exceeding 25% and plasticity index not exceeding 6.

(ii) For surfacing : Liquid limit not exceeding 35% and plasticity index between 4 and 9.

If the soil from one source does not meet the gradation and plasticity requirements of a job, it becomes necessary to mix materials from two more sources for obtaining the desired mixture. The blending of materials can be carried out by making trial combinations.

Proper compaction plays a very important role in stabilisation. Compaction has a great effect on soil properties, such as strength and stress-strain characteristics, permeability, compression, swelling and water absorption. The properties of a soil under compaction depend upon the water content, amount of compaction and the type of compaction. Compared to coarse grained soils, the properties of fine grained soils are affected to a greater extent by the placement conditions. Compaction has been discussed in Chapter 17.

### 31.3. CEMENT STABILISATION

1. **Soil cement and its influencing factors.** The soil stabilised with cement (Portland) is known as *soil cement*. The cementing action is believed to be the result of chemical reaction of cement with the silicious soil during hydration. The binding action of individual particles through cement may be possible only in coarse-grained soils. In fine grained, cohesive

soils, only some of the particles can be expected to have cement bonds, and the rest will be bonded through natural cohesion.

The important factors affecting soil cement are : nature of soil, cement content, conditions of mixing, compaction and curing, and admixtures :

(i) *Nature of soil.*

Almost every inorganic soil capable of pulverisation can be successfully stabilised with cement, although the cement requirement will increase with the increase in the specific surface of a soil. Expansive soils are particularly difficult to be stabilised. Organic matter and sodium sulphate are harmful and weaken the soil-cement. Well-graded soils, with less than 50% fraction finer than 75-micron sieve and plasticity index less than 20, are found to give best results. Fig. 31.1 shows the approximate gradation limits of soil for economic stabilisation with cement (HRB, 1943).

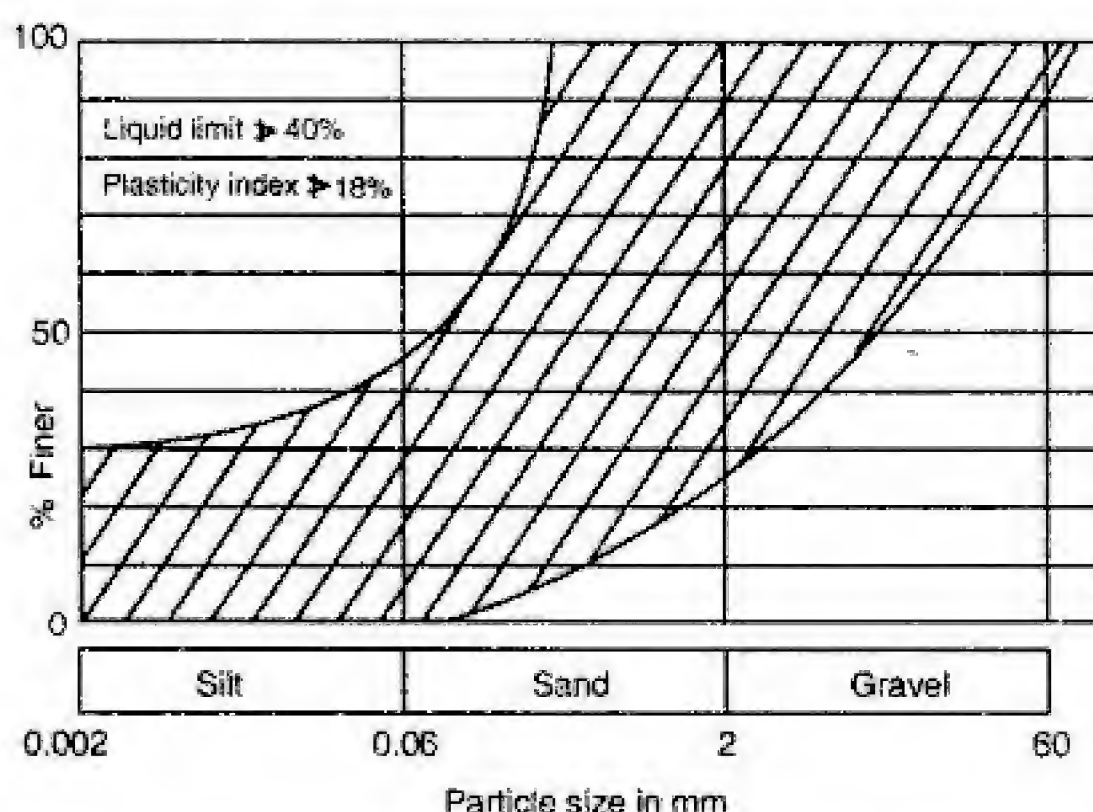


FIG. 31.1. APPROXIMATE GRADATION LIMITS FOR SOIL-CEMENT.

(ii) *Cement content.*

The strength of soil cement increases with an increase in the amount of cement added to a soil, and if such an increase in strength does not result, the soil may normally be considered unsuitable. The ordinary Portland cement is generally used for stabilisation. High-early-strength cement can also be used, and is usually more effective than normal cement (Lambe, 1962).

The amount of cement required, expressed as a percentage by weight of dry soil, generally varies between 5 to 15%, finer soils requiring more cement. The actual amount required is determined experimentally by performing compressive strength and durability tests. The amount of cement giving a compressive strength of 25 to 30 kg/cm<sup>2</sup> should normally prove satisfactory for tropical climates. The approximate amounts needed for different types

of soils may be as follows : gravels, 5 to 10% ; sands, 7 to 12% ; silts, 12 to 15% and clays, 12 to 20%.

If a layer of soil cement having a surface area  $A$  ( $\text{m}^2$ ), thickness  $H$  (cm) and dry density  $\gamma_d$  ( $\text{t}/\text{m}^3$ ) is to be stabilised with  $p$  percentage of cement on the basis of dry weight of soil, the percentage of cement by weight on the basis of dry soil cement mixture will be  $100p/(100 + p)$ , and the amount of cement needed in tonnes for the construction will be given by the expression :

$$\text{Amount of cement} = \frac{AH\gamma_d}{100} \cdot \frac{p}{100 + p} \quad \dots(31.1)$$

The number of cement bags, weighing 50 kg each, will be given by :

$$\text{Number of cement bags} = \frac{AH\gamma_d p}{5(100 + p)} \quad \dots(31.2)$$

### (iii) *Mixing, compaction and curing.*

A stronger and more durable soil-cement, will be produced, if the soil-cement water mixture is more intimately mixed. Mixing will, however, result in decreased strength if it is continued long after the cement hydration has begun.

The amount of water to be added is decided from the considerations of good compaction and this amount is considered adequate enough for cement hydration also. It has been observed that better strength and stability develops, if the fine grained soils are compacted wet of optimum water content and the coarse grained soil on the dry side of the optimum water content (Lambe, 1962). The greater is the compacted density, the stronger and more durable will be the soil cement.

Like concrete, the strength of soil-cement increases with age. Hence it should be moist cured for at least 7 days. Curing is rapid at higher temperatures.

### (iv) *Admixtures.*

Certain chemicals are sometimes added to soil-cement, with the purpose either to reduce the cement consumption or to make a soil suitable for stabilisation which is not responsive to cement alone in its natural state. Lime and calcium chloride are commonly used with clays and soils containing organic matter. Sodium carbonate and sodium sulphate have also been tried. Fly ash, as an additive, has also been found very beneficial in the stabilisation of dune sand (Davidson, *et. al*, 1958). The fly ash acts as a puzzolana and also as a filler for increasing the density (Uppal and Bhalla, 1965).

## 2. *Construction methods.*

The normal construction sequence for soil-cement bases is as follows : (i) shaping the sub-grade and scarifying the soil, (ii) pulverising the soil, (iii) adding and mixing cement, (iv) adding and mixing water, (v) compacting, (vi) finishing, (vii) curing and (viii) adding wearing surfacing. There are three methods of carrying out these operations : (i) mix-in place method, (ii) travelling plant method and (iii) stationary plant method.



In the *mix-in-place* method, the sub-grade is first shaped to the required grade and is cleared of undesirable materials. It is then scarified to the required depth of treatment and the soils is pulverised, until at least 80% of the material (excluding stones) passes a 4.75 mm sieve. If another soil is to be blended, it is mixed with the loose, pulverised soil. The pulverised soil is spread and shaped to proper grade. Calculated amount of cement is then evenly distributed over the surface and intimately mixed. Water is added as required for compaction and the soil cement-water is turned into an intimate mixture. The wet mixture operation should not last more than 3 hours, after which the compaction should be completed within the next 2 hours. It is fairly easy to process coarse grained soil. Pulverisation and mixing of plastic clays can be facilitated by adding lime in proportions of 1 to 4%. The compacted soil cement is moistured for at least 7 days. A bituminous wearing surface is normally provided to protect the soil-cement base from abrasion and absorption of water in shrinkage cracks.

The mix-in-place method is considered cheaper and more adaptable to different field conditions, but the processing of soil is not so thorough and accurate as with other methods.

In the *travelling plant method*, the pulverised soil is heaped into a windrow and the cement is spread on the top. The soil and cement are lifted by an elevator to a mixer carried on a travelling platform where water is added and mixing is done. The mixture is then discharged on to the subgrade. It is spread with a grader and compacted. Uniform mixing and accurate control on added water can be ensured in the method. A uniform subgrade surface with controlled depth of treatment is possible. The plant is however, costly. In the *stationary plant method*, the excavated soil is brought to a stationary mixing plant. At the plant, cement and water are added and mixed with the soil. The mixture is then transported back to the desired location, dumped, spread and compacted. Similar to the travelling method, the method affords an accurate proportioning of materials and thorough mixing. The depth of treatment can be easily controlled. The method is slower and may prove expensive due to additional haulage of soil.

#### 31.4. LIME STABILISATION

Hydrated (or slaked) lime is very effective in treating heavy, plastic clayey soils. Lime may be used alone, or in combination with cement, bitumen or fly ash. Sandy soils can also be stabilised with these combinations. Lime has been mainly used for stabilising the road bases and sub-grades.

On addition of lime to soil, two main types of chemical reactions occur : (i) alteration in the nature of the absorbed layer through base exchange phenomenon, and (ii) cementing or pozzolanic action. Lime reduces the plasticity index of highly plastic soils making them more friable and easy to be handled and pulverised. The plasticity index of soils of low plasticity generally increases. There is generally an increase in the optimum water content and a decrease in the maximum compacted density, but the strength and durability increases.

The amount of lime required may be used on the unconfined compressive strength or the CBR test criteria. Normally 2 or 8% of lime may be required for coarse grained soils, and 5 to 10% for plastic soils. The amount of fly ash as admixture may vary from 8 to 20% of the soil weight (Lambe, 1962).

The construction procedures of lime stabilised bases are similar to those soil-cement. No strict time limitations for completion of the job are however necessary, since the soil-lime cementation reactions are respectively slow.

### 31.5. BITUMEN STABILISATION

Asphalts and tars are the bituminous materials which are used for stabilisation of soil, generally for pavement construction. These materials are normally too viscous to be incorporated directly with soil. The fluidity of asphalts is increased by either heating, emulsifying or by cut-back process. Tars are heated or cut back. The bituminous materials when added to a soil impart cohesion or binding action and reduced water absorption. Thus either the binding action or the water proofing action or both the actions, may be utilised for stabilisation. Depending upon these actions and the nature of soils, bitumen stabilisation is classified under the following four types : (i) sand- bitumen, (ii) soil-bitumen, (iii) water-proofed mechanical stabilisation and (iv) oiled earth.

#### 1. Sand bitumen.

This term refers to bitumen-stabilised cohesionless soil, such as loose beach, dune, pit or river sand. The primary function of bitumen is to bind the soil particles. Sand should be substantially free from clay and organic matter. The gradation may vary within a wide range, but the fraction passing a 75 micron sieve should normally not exceed 12% ; in case of fine dune sand the fraction may be upto 25%. Crushed stone, rock dust, gravel, etc., may be added to poorly graded sand.

The climatic conditions such as rainfall and temperature decide the type of bituminous material to be used and the method of mixing and construction to be employed. Hot mix sand asphalt is suitable in area of heavy rainfall, and emulsions are preferable in arid zones. Rapid curing cut-backs are recommended for low temperatures and slow curing for high temperatures. The quantity of bituminous material required is determined by laboratory tests. The approximate proportions on dry weight basis of sand are as follows (Uppal and Bhalla, 1965); hot mix asphalt, 5 to 11% ; cutback, 4 to 10% ; emulsions, 5 to 10%. Hydrated lime, 1 to 2 % is sometimes used as an admixture to assist coating of sand grains.

#### 2. Soil bitumen.

It refers to a cohesive soil in which the main function of bitumen is to preserve the natural cohesive strength by water-proofing the soil or reducing the water absorption. A large variety of soils can be thus stabilised. For best results the following requirements are recommended (HRB, 1946) :

- (i) Maximum size : not greater than approximately one-third the compacted thickness,
- (ii) Passing 4.75 mm sieve : more than 50% ,
- (iii) Passing 425 micron sieve : 35—100%,
- (iv) Passing 75 micron sieve 10—50%,
- (v) Liquid limit : Less than 40%,
- (vi) Plasticity index : Less than 18.

**PART VIII**

**MISCELLANEOUS TOPICS**

- 32. SITE INVESTIGATION AND SUB-SOIL EXPLORATION**
- 33. ADVANCED MEASURING INSTRUMENTS**
- 34. GEOTEXTILES**



# Site Investigation and Sub-Soil Exploration

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## 32.1. INTRODUCTION

An investigation of site is essential for judging its suitability for the proposed engineering works and for preparing adequate and economic designs. Site investigation is equally necessary for analysing the safety or causes of failure of existing works, for selecting construction materials and for deciding upon the construction methods to be applied. In general, the purpose of a site investigation is to obtain necessary information about the soil and hydrological conditions at the site and to know the engineering properties of soil which will be affected. A timely and intelligently planned site investigation should be considered a pre-requisite for efficient, safe, economical design and construction.

The methods of site investigation are largely dependent upon the nature of the engineering project and the site. In general, any investigation should start with the collection and examination of the already existing data about the soil and geological conditions of the site. In many areas, the existing local knowledge, records of trial pits, bore holes, etc., in the vicinity and the behaviour of existing structures, particularly if they are similar to the proposed ones, are very helpful. If the existing information is not sufficient or is inconclusive, the site should be explored in detail. The exploration should be preceded by site reconnaissance.

## 32.2. SITE RECONNAISSANCE

An inspection of the site and study of topographical features is often helpful in getting useful information about the soil and ground water conditions and in deciding the future programme of exploration. On going over the site, a study of the following features may be useful : local topography, excavations, cuttings, quarries, escarpments, evidence of erosion on land slides, fills, water levels in wells and streams, flood marks, and drainage pattern, etc. If there has been an earlier use of the site, information should be gathered in particular about the underground workings, if any, and about the location of fills and excavations.

## 32.3. SITE EXPLORATION

The object of site exploration is to provide reliable, specific and detailed information about the soil and ground water conditions of the site which may be required for a safe



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in a grid pattern and core penetration or other sounding tests may be performed at every 100 metres. The number of pits and borings is then decided by examining the variation in the penetration curves.

On dam sites, preliminary borings may be made at 50 m spacings along the top line of the upstream face of the dam and across one or both abutments. A few widely scattered holes are also required in the bottom of the reservoir on the *u/s* side of the dam. Depending upon the subsoil conditions the number and disposition of bore holes are varied, usually in the valley floor, on the abutments and in the locations of appurtenant structures. On road sites, the pits or bore holes are usually made along the proposed centre line of the road and along the ditch lines, alternating right and left ditches. The spacing may be about 100 m to start with which may be increased to 500 m on obviously uniform soils or decreased to 30 m or less, if frequent changes in the soil profile are met.

**General exploration :** Depending upon the details or the precision of the sub-soil data to be obtained, the site exploration may be classified as *general* (or preliminary) and *detailed*. The primary aim of a general exploration is to get an approximate picture of the sub-soil conditions at a relatively low cost. The information so obtained should suffice for the design and execution of minor and routine engineering works. General exploration is also recommended on erratic soil profiles where a detailed exploration may prove very expensive. Exploratory borings, shallow test pits, subsurface penetration and soundings, and geophysical methods are used for general explorations. Exploratory borings are carried out by auger, auger and shell, and by wash boring. Disturbed, or partly disturbed, but representative samples collected from test pits and borings are subjected to simple laboratory and field tests, such as the identification and classification, *in-situ* water content, density and unconfined compressive strength. Penetration and soundings help in estimating the relative density and strength properties of coarse-grained soils and also to locate the weaker strata and lenses. The general exploration is thus able to give information about the following :

1. Depth, extent and composition of soil strata.
2. Depth of rock, when necessary.
3. Ground water level.
4. Engineering properties requiring disturbed or partly disturbed samples.
5. Approximate values of strength and compressibility of soil strata.
6. Preliminary selection of foundation type.

**Detailed exploration :** Detailed exploration follows as a supplement to general exploration when large engineering works, heavy loads, and complex and costly foundations are involved. A detailed exploration is meant to furnish information about soil properties such as shear strength, compressibility, density index, and permeability. These properties are known by procuring and testing undisturbed samples and by field tests. The field test may be conducted for determining the *in-situ* values of density, bearing capacity, shear strength, permeability, and pore pressures.

**Ground water levels :** The location of ground water level is important for deep excavation and foundation works, and also on sites liable to be water-logged. Open wells



is kept closed with the piston. When the desired sampling elevation is reached, the piston rod is clamped, thereby keeping the piston stationary, and the sampler tube is advanced down into the soil. The sampler is then lifted up, with piston rod in the clamped position. The piston prevents the entry of water and soil into the tube, when it is being lowered, and then greatly helps to retain the sample during lifting operations. Thus the sampler is more suitable for sampling soft soils and saturated sand.

**3. Rotary samplers.** These are the core barrel types (Earth Manual, 1960) having an outer tube provided with cutting teeth and a removable thin wall liner inside. It is used for firm to hard cohesive soils and cemented soils.

Fig. 32.5. shows a few typical samplers.

### 32.6. DISTURBED SAMPLING

Disturbed samples can be obtained by direct excavations, augers and thick wall samplers. For sampling saturated cohesionless soils, a trap valve or a spring sample retainer is inserted in the drive shoe (cutting edge). Sand pump and shell with trap valve (flap) are also used. The disturbed samples may be used for mechanical analysis, water content determination, index properties tests, compaction and stabilization tests. The split spoon samplers can be used for approximate determination, of unconfined compressive strength. The samplers should be so transported and stored that the original composition is preserved and the water content also does not change, if desired.

### 32.7. UNDISTURBED SAMPLING

Undisturbed samples may be required for tests on shear, consolidation, and permeability. They can also be used for other tests like the disturbed samples.

Undisturbed samples are obtained by forcing a thin wall sampler into the soil at the bottom of the bore hole or in a test pit. The penetration of the sampler into the soil should be continuous and rapid. The sampler should never be over driven so as to compress the sample. A piston sampler may be used with advantage in soft soils, specially below water table.

Undisturbed samples of cohesionless soils, specially from below the water table are difficult to be obtained. A compressed air sampler may be used. It enables the sample to be removed from the ground into an air chamber and then lifted to the ground surface without contact with water of the drill hole. A piston sampler with bore holes kept filled with drilling mud can also be used. Alternative methods may be to impart cohesion to sand by asphaltic emulsions, or to freeze the sand at the sampling depth or near the lower end of the sampler.

It is relatively easier to obtain undisturbed samples of both cohesive and cohesionless soils from open test pits and trenches. Sharp-edged thin wall tubes or cylinders can be used for this purpose. Alternatively, *hand cut samples* can be obtained. In stiff and sufficiently cohesive soil, undisturbed samples can be cut out and trimmed to regular shapes, say a cube. The sample is wrapped in wax paper or polythelene sheet and is given a wax coating on the outside. A sample may also be cut by removing the soil from the sides and left projecting like a stump. A wooden box or other suitable container with



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## Advanced Measuring Instruments

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### 33.1. ELECTRONIC CONSOLIDATION APPARATUS

Electronic one dimensional consolidation apparatus basically consists of the usual loading frame with weights, consolidation cell with porous stones, pressure pad, cutting ring, water reservoir etc., as per conventional unit, but with a sensitive displacement transducer in the place of the usual displacement dial gauge. The sensitive displacement transducer, having range of 0 to 10 mm is connected to a single/multi-channel read out unit to sense the signals received from the transducer. The consolidation of the specimen is directly displayed at selectable time interval. Plate XI shows the photograph of an electronic consolidation apparatus (bench type, front loading) with read out unit, manufactured by HEICO (Hydraulic and Engineering Instruments, New Delhi). The unit has the provision to be coupled to the *data-logger* to simultaneously display and print out the readings at pre-programmed time intervals.

### 33.2. ELECTRONIC DIRECT SHEAR APPARATUS

Plate XII (a) shows an electronic direct shear apparatus manufactured by HEICO. In addition to other basic components like shear box assembly, gripper assembly, porous stones, shear box housing, loading units for application of normal load to the specimen according to the requirements as per conventional units, the electronic unit is fitted with two nos. LVDT'S—one for consolidation measurement and the other for shear strain measurement, one load cell (electronic) as per the capacity of the machine for measuring shear force and a three channel read out unit which can be connected to the two displacement transducers and the load cell for measuring consolidation, horizontal strain and shear load respectively in their direct engineering units. The system is compatible for connection to a programmable micro-processor based data-logger for continuous display and printing of the test readings at pre-set time intervals and sequences.

Plate XII (b) shows a HEICO multi-channel Data-Logger designed around the single chip micro-processor, incorporating latest electronic circuit. It can serve as a common module for other electronic instruments such as triaxial shear, direct shear and consolidation test. This can be used for all the tests running simultaneously, or for any one test with the help of channel skip facility, incorporated in the unit. In addition to continuous display of the different parameters directly in their respective engineering units, in its display screen,

they can be printed by the numerical printer alongwith time and channel no., at programmed time intervals and sequences. The system can be programmed to shift the decimal points according to requirement. The data-logger has provision of peak hold facility which stores the peak values of any parameter, attained during the test.

### 33.3. ELECTRONIC TRIAXIAL SHEAR TEST EQUIPMENT

Plate XIII shows HEICO electronic triaxial test apparatus. This instrument eliminates the constant recording and computation of the data by the observer. All the above functions are carried out by continuous sensing of pore pressure, axial strain and axial load with the help of pore pressure transducer, displacement transducer and load cell respectively and their display directly in respective engineering units on the digital read out unit coupled to it through selectable switch. The unit is designed to be connected to micro-processor based data-logger [Plate XI (b)] which continuously scan the data received from the transducer and in addition to their display, prints them in their respective engineering units at programmed time intervals and sequences. A battery is provided to store the data programmed into the data-logger during power failure.

### 33.4. HYDRAULIC EARTH PRESSURE CELL

Earth pressure cells are used to measure the actual earth pressure on retaining walls, building basements, bridge abutments sheet piling surface of tunnel lining as well as for measurement of total pressure at foundations of earth dams and embankments. Such measurements help in evaluating their post construction behaviour and taking timely remedial measures for the structures showing distress.

Earth pressure cells, also called *stress cells*, are generally of two categories :

- (i) Flexible diaphragm type, and
- (ii) Stiff cylinder type.

The *flexible diaphragm type* earth pressure cell consists of a flexible circular or rectangular diaphragm attached to a rigid stiff case. The pressure is measured due to continuous displaced shape of the flexible diaphragm, the greatest deflection occurring at the centre. In the *stiff cylinder type cell*, the axial compression of the stiff, prismatic element, usually enclosed within a case to isolate it from the lateral stresses of the surrounding soil mass, is used to sense the total pressure.

Various systems available to measure earth pressure use the following :

- (i) Electrical resistance strain gauges
- (ii) Semi-conductor strain gauges,
- (iii) Vibrating wire system,
- (iv) Closed fluid system (usually called Gloetz or *hydraulic pressure cell*), and
- (v) Pneumatic system, where air pressure is used to balance the stiffness of the cell.

Out of the above systems, the strain gauge type, the vibrating wire type and the closed fluid system type (*i.e* hydraulic pressure cell) are commonly used since they are most accurate. Resistance strain gauge type cells are easy to use and have linear rapid response, but they are susceptible to damage and are affected by the moisture of the earth fill material in long use. Vibrating wire type cells are more durable but have non-linear response ; these are described in § 33.5.



The hydraulic earth pressure cell, also known as Gloetz cell, has recently come into vogue as means of measuring total pressure changes in soil, earth or rockfill, or at the interface between any of these materials. It may also be used to measure pressure changes in rock, when installed in a machined slot.

Plate XIV shows the photograph of a hydraulic pressure cell manufactured by HEICO. The pressure cell essentially consists of a sensor flat jack or fluid filled pressure pad connected to a hydraulic or pneumatic diaphragm transducer, which in turn, is connected by a flexible tubing to a read out unit. Pressure transferred from the surrounding soil to the flat jack is measured by balancing the fluid pressure in the cell by a pressure applied to the reverse side of the transducer diaphragm.

The flat jack (pressure sensor or cell) is formed from two sheets of stainless steel welded around the periphery. The narrow gap of 1.5 mm between the plates is filled with fluid of comparable deformity to that of the ground. Mercury is used in rocks and oil is used in soils. The cells can be either circular or rectangular in plan with dimensions ranging from 6 to 40 cm.

The cell is connected to a hydraulic transducer by a short length of stainless steel tubing forming a closed hydraulic circuit. Both the cell and the transducer are embedded in the structure to be monitored. Hydraulic transducer is a hydraulic valve consisting of a flexible steel, plastic or rubber diaphragm, incorporated in metal housing. The diaphragm must completely separate the cell fluid from the measuring fluid. One side of the diaphragm is connected to the cell fluid and the other to the measuring fluid delivery and return tubes. The transducer design is such that the pressure in cell fluid is slightly greater than that in the measuring fluid in order to prevent return of the measuring fluid. When the applied measuring pressure equals the cell fluid pressure, the diaphragm will displace, allowing flow along the measuring fluid return line.

Read out equipment consists of a fluid reservoir, a pump with pressure gauge to measure the applied pressure, and a detector to indicate the fluid return from the cell. Smaller applied pressures are measured by using a manometer instead of pressure gauge.

Hydraulic transducer is connected to the terminal panel or directly to the read out unit by two nylon high pressure pipes carrying quick couplings at each end. To take reading, hydraulic pressure is supplied from the read out unit to one side of the flexible diaphragm valve incorporated in the transducer. When the supply

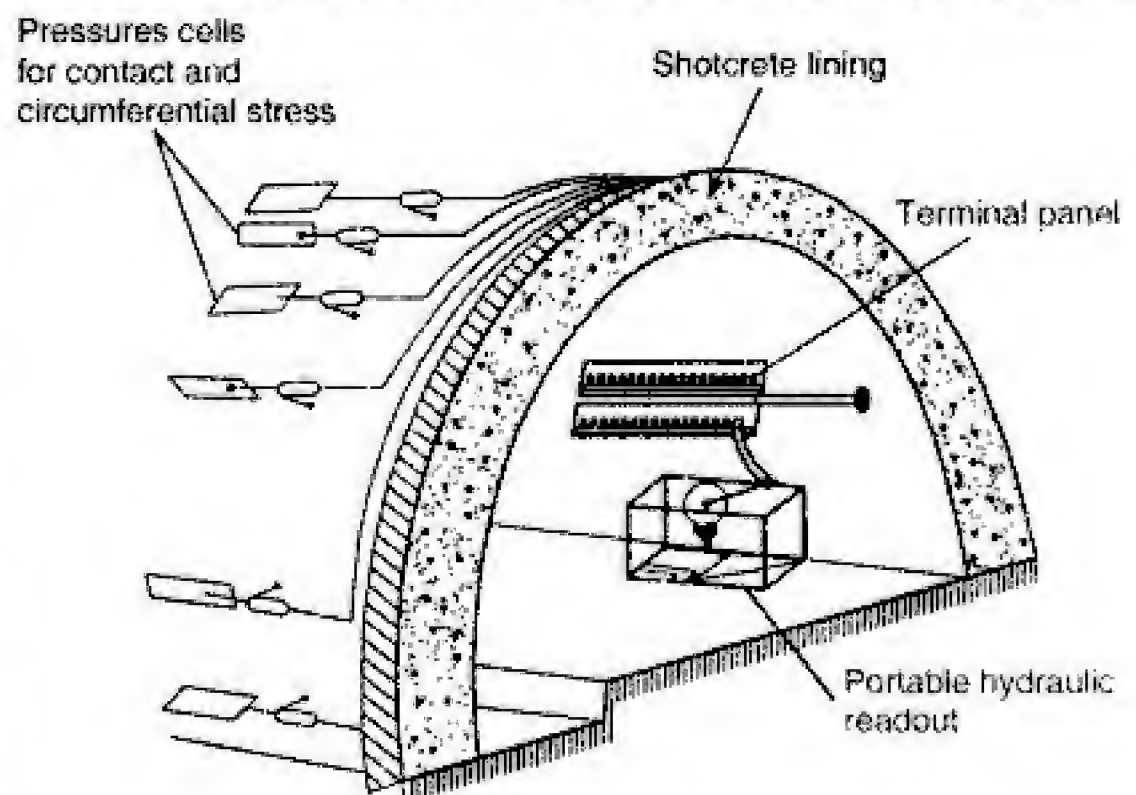


FIG. 33.1 INSTALLATION OF CELLS IN TUNNEL LINING

pressure is sufficient to balance the cell pressure on the reverse side of the diaphragm, the valve opens, allowing flow along the return line to a detector in the read out unit. The balancing pressure is read on the bourden type pressure gauge.

Cells are generally installed in pairs or clusters, to measure in different directions at the same location. Adjacent cell should be separated by a distance of atleast one cell diameter in such a way as to prevent the presence of a cell affecting reading on adjacent cells. Typical layouts are shown in Fig. 33.1.

When installing cells in a natural soil or fill embankment, an excavation is made to accommodate a cell cluster, then individual pockets for each cell are hand dug at the correct locations with flat faces at the required inclinations. When placing cells adjacent to piers, piles, retaining walls, culverts and other structures, the cells may either be attached to the form work and placed in the structure, or fastened to the backfill, a short distance away from the structure. The contact between the cell and the backfill material should be effected by means of a layer of fine grained material.

### 33.5. VIBRATING-WIRE EARTH PRESSURE CELLS

Vibrating-wire earth pressure cell was originally developed and used for earth pressure measurements on sheet piles for strutted excavations. The primary requirement was a pressure cell that was robust enough to tolerate moderate pile driving and stable enough to permit long term measurements. Subsequently, these cells have been used to measure total pressure and/or pore pressure (with filter) acting on sheet piles, retaining walls foundations, tunnels and silos, as well as stresses in earth dams. These are also widely used in earth/rock fill dams to measure total earth pressure.

Plate XV (a) shows vibrating wire type earth pressure cell manufactured by AIMIL (India) in collaboration with GEONAR A/S, Norway. The principle of operation of a vibrating wire earth pressure cell or transducer is depicted in Fig. 33.2.

When an external pressure is applied to the membrane, to which the stretched *vibrating-wire* is rigidly attached, it causes a change in tension in the wire due to the deflection of the membrane posts. This causes a change in the stress in the gauge-wire and hence a change in the natural frequency of vibration. Consequently, the change in the frequency of the gauge wire vibrations is a measure of the change in external pressure acting on by the membrane surface. The frequency signal is transmitted by the *electrial cable* to a *read out instrument*. The change in earth pressure acing on the membrane surface is proportional to the difference of the squared frequency of vibration for the corresponding pressures.

The output of a vibrating wire transducer is an alternating current which has the same frequency as the natural frequency of vibration of the gauge wire. The only measurement required is to determine to frequency of the alternating current. Once the frequency is

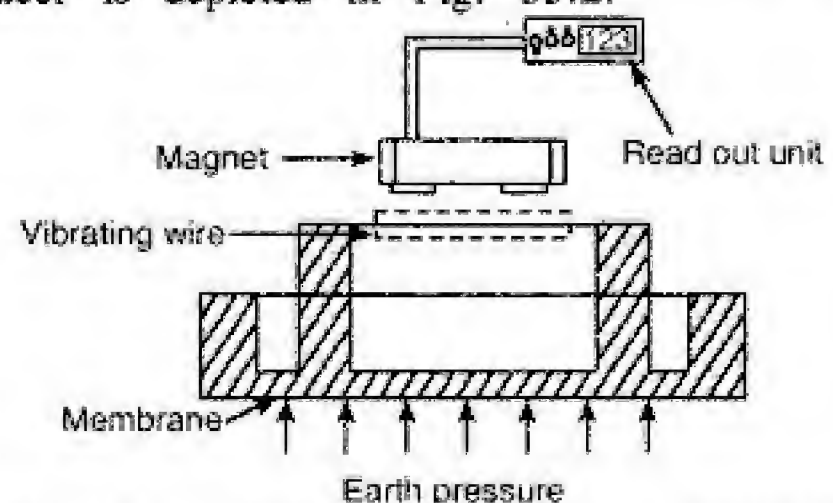


FIG. 33.2. VIBRATING-WIRE EARTH PRESSURE TRANSDUCER.



known, the magnitude of the physical quantity which is measured, such as pressure, strain or force, can be calculated using formula and constants in calibration schemes supplied with each transducer.

All vibrating-wire type transducers contain an electrical pick-up and exciter circuit as shown in Fig. 33.3. The electrical components, which are potted in epoxy, are mounted inside the body of the transducer.

AIMIL manufactures two types of cell : P-100 cell and P-105 cell. The active membrane diameter of P-100 cells (75 mm) limits the grain size of the soil to 1.5 mm. For larger grain size (0–2 mm), in earth/rock fill dams and off-shore applications, the P-105 cell with its active membrane diameter of 100 mm is recommended. Because of their thickness/diameter ratio (0.28 for P-100 and 0.22 for P-105), the transducers are installed in special steel frames when used in earth fills for earth pressure measurements.

The P-100 transducer is often used to measure total pressure or pore pressure acting on different kinds of walls. For low pressures, atmospheric pressure may be maintained inside the transducer through the *PE* tube encasing the lead wires. The signal cable P-540 is used alongwith this cell.

The P-540 cable is made of standard *PPOP* one-pair shielded cable with *OD* 10 mm *PE* tubing outside as an extra protection and to maintain atmospheric pressure inside the transducer.

The P-105 transducer is widely used in earth/rockfill dams to measure total earth pressure. Plate XV (c) shows vertical positioned P-105 cell in dam embankment. Gauge redundancy has been provided for by installing two independent vibrating-wire systems (wire and magnet system) inside the transducer. The P-105 transducer is not vented to atmosphere ; hence variations in atmospheric pressure should not be ignored at low total pressures. The signal cable P-430 is used with this cell.

The P-430 cable is specially designed to withstand strong vertical forces. It is steel armoured and has a thick *PE* outside insulation (Fig. 33.5). The space

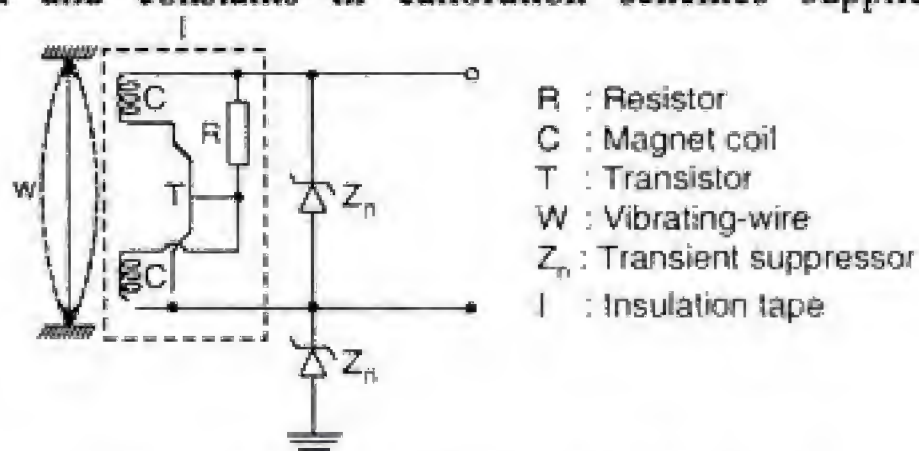


FIG. 33.3. SCHEMATIC DIAGRAM OF VIBRATING-WIRE STRAIN GAUGE (AIMIL).

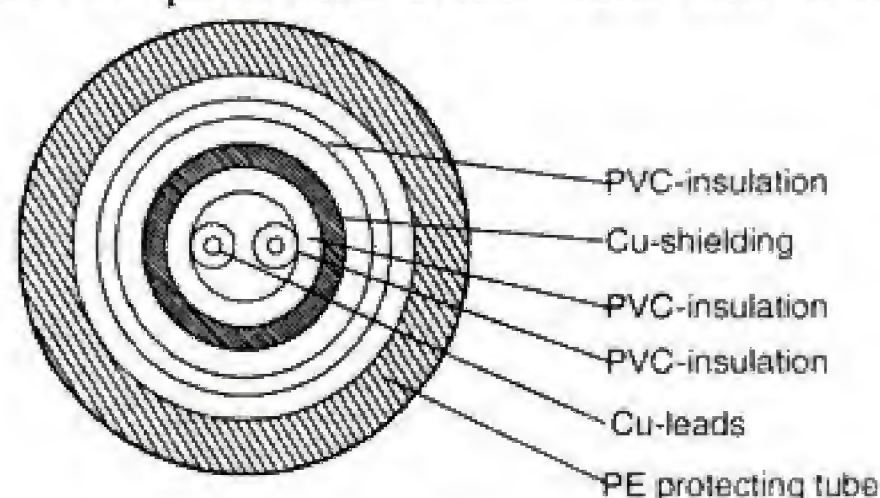


FIG. 33.4. P-540 CABLE.

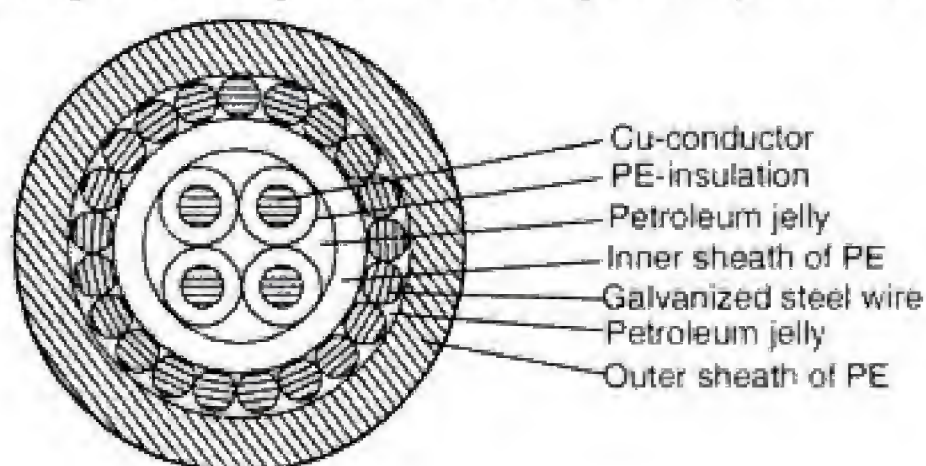


FIG. 33.5. P-430 CABLE.



between the cable cores and also between steel armour wire are filled with petroleum jelly to prevent longitudinal leakages caused by light surface damage to the cable. An extra protecting PE tubing for the P-430 cable (OD  $25 \times 3$  mm) is sometimes used in adverse environments and where large settlements are expected.

Reading of P-100 and P-105 transducers may be performed either manually or the automatic recording equipment. Plates XV (a) and XVI show the P-520 F frequency indicator. It may be connected directly to the transducer cable or to a switch box. Automatic read out can be obtained by means of a micro-logger with digital print out of all data in engineering units.

### 33.6. VIBRATING-WIRE EXTENSOMETER

A vibrating-wire extensometer is used to monitor displacements or strains in earth structures. It is particularly useful for monitoring internal deformation and cracking of dam embankment near the abutments.

The extensometers are normally linked together with steel pipe and anchor plates to form a continuous chain over the distance to be monitored. Fig. 33.6 shows P-265 extensometer manufactured by AIMIL.

The complete measuring system consists of three main parts :

- (i) Extensometer
- (ii) Cable
- (iii) Read out instrument.

The basic principle (Fig. 33.7) of the vibrating-wire strain gauge is that the change in natural frequency of a stretched wire depends on the change of the tension in the wire. In this instrument one end of the wire is attached to the movable head of the extensometer by a steel spring. A displacement of the extensometer is thus transformed to a variation in tension of the spring and also in the vibrating-wire. Thus, the frequency of the wire is a measure of the displacement between extensometer and anchor. The square difference of frequencies is proportional to the displacement. The signal cable of P-430 type (Fig. 33.5) is used. The P-430 cable is specially designed to withstand strong external forces in hostile environments.

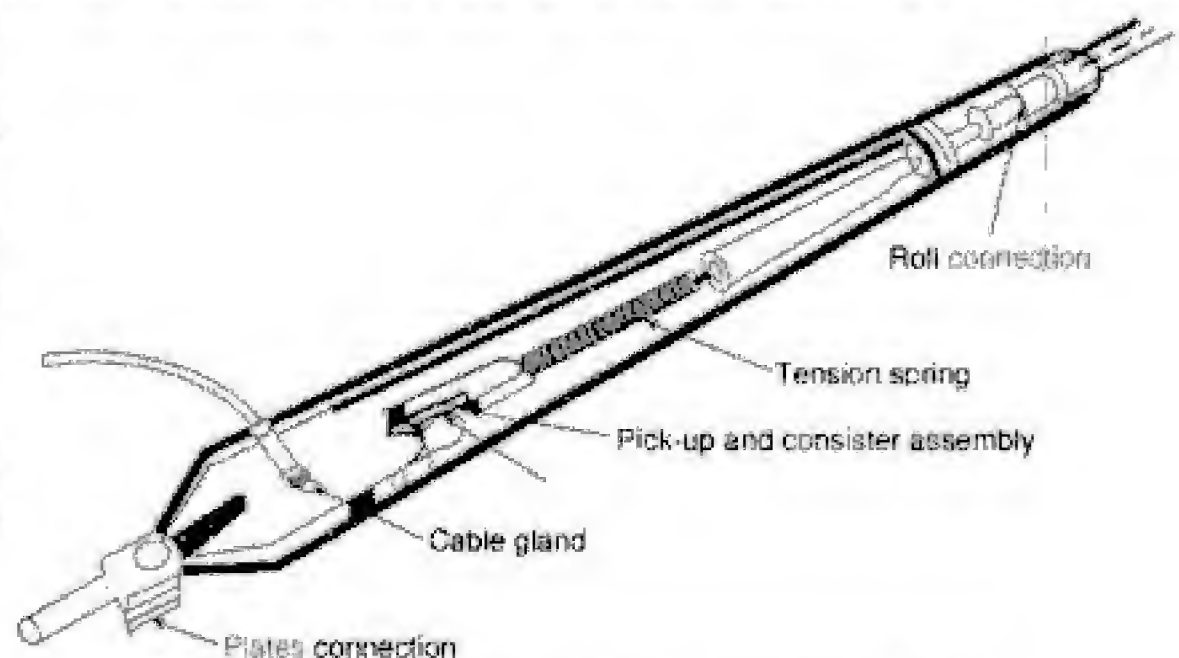


FIG. 33.6. VIBRATING-WIRE EXTENSOMETER.

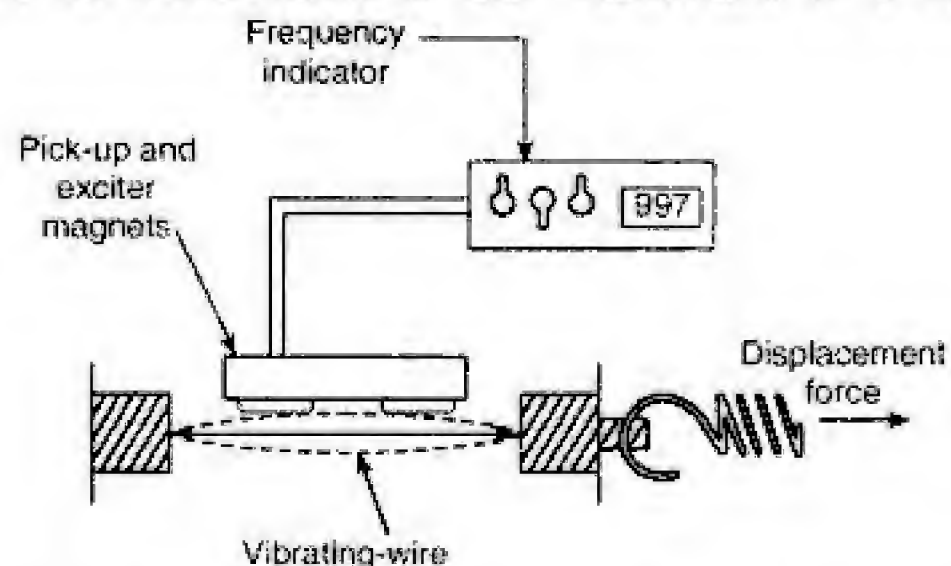
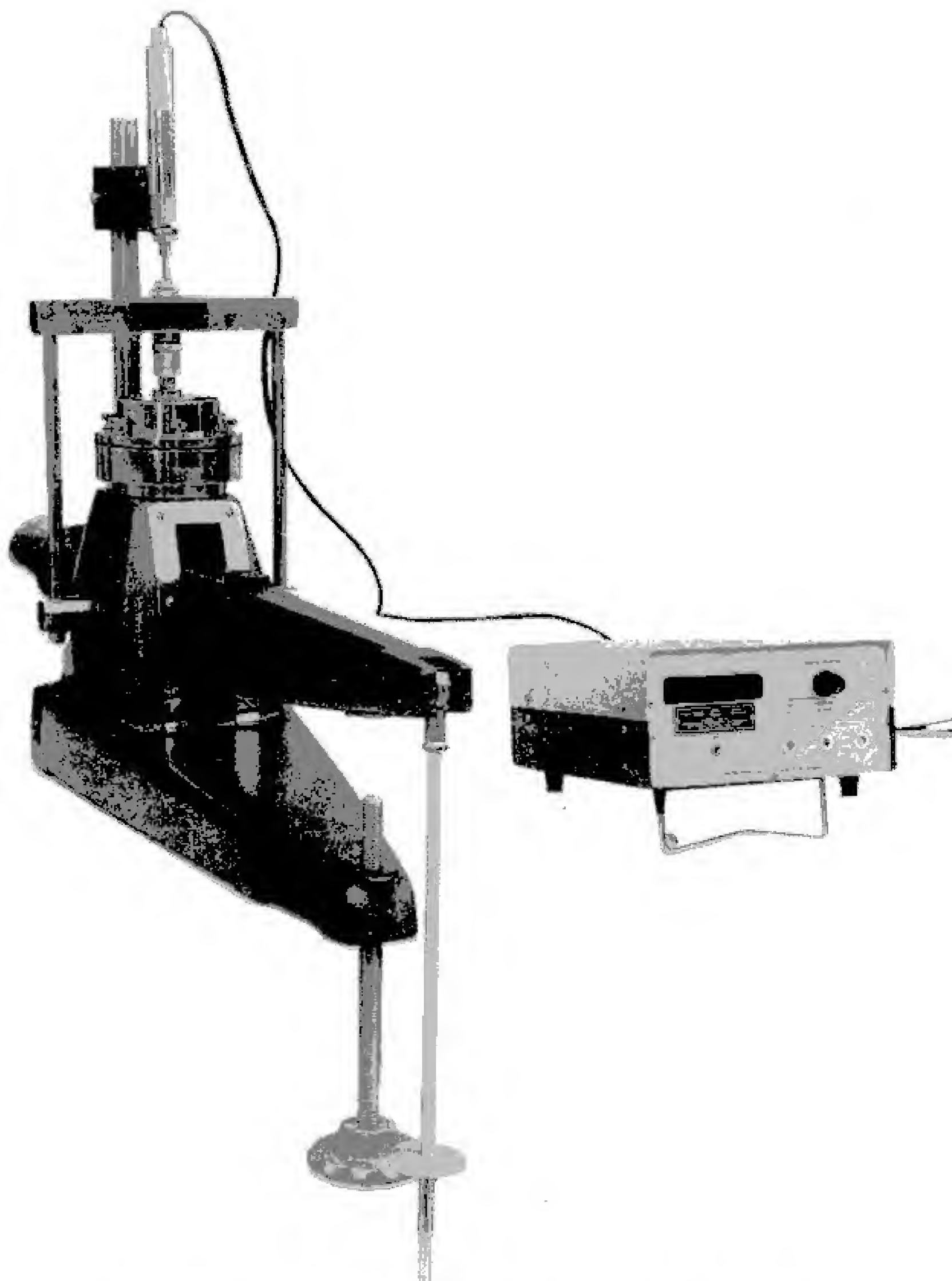


FIG. 33.7. SCHEMATIC DIAGRAM OF VIBRATING-WIRE EXTENSOMETER.

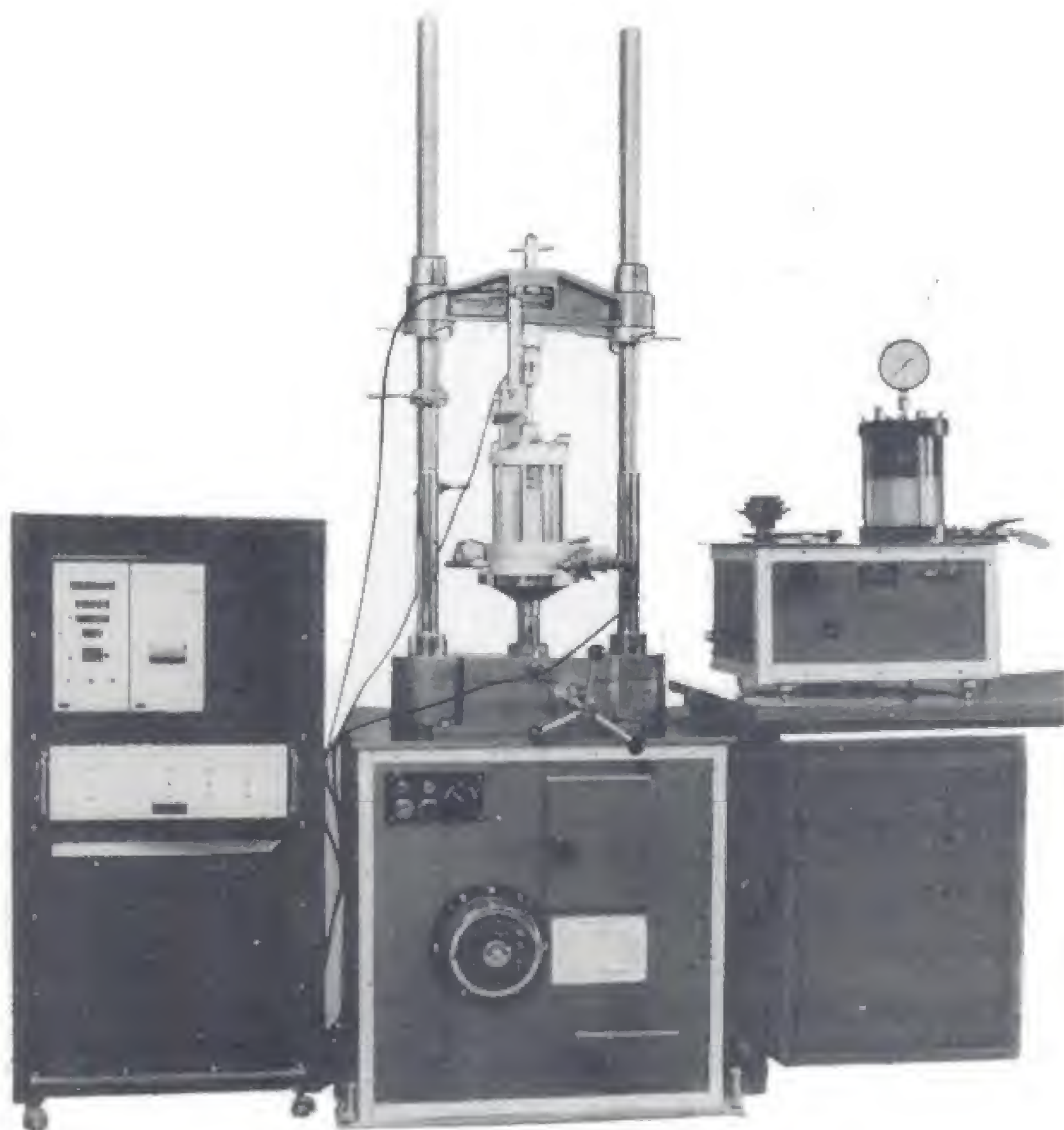


ELECTRONIC CONSOLIDATION APPARATUS WITH READ-OUT UNIT (HEICO)

PLATE XI

(To Face Page 880)

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**ELECTRONIC TRIAXIAL TEST APPARATUS WITH DATA LOGGER (HEICO)**

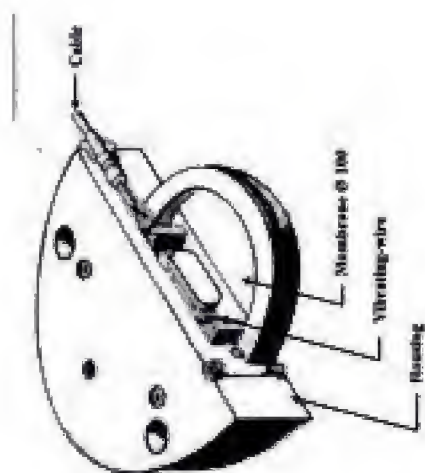
**PLATE XIII**

(To Face Page 880)





(a) VIBRATING WIRE EARTH PRESSURE CELL WITH CABLE AND READOUT UNIT

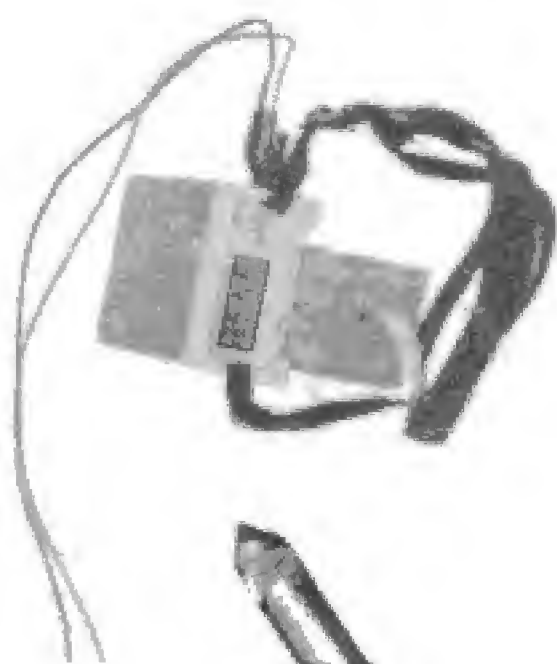


(b) CELL INTERIOR



(c) P105 CELL IN EARTH EMBANKMENT

(AIMIL/GEONAR)  
PLATE XV



E AND READOUT UNIT

stress, voids ratio. In addition, the system automatically computes the coefficient of consolidation for each load increment.

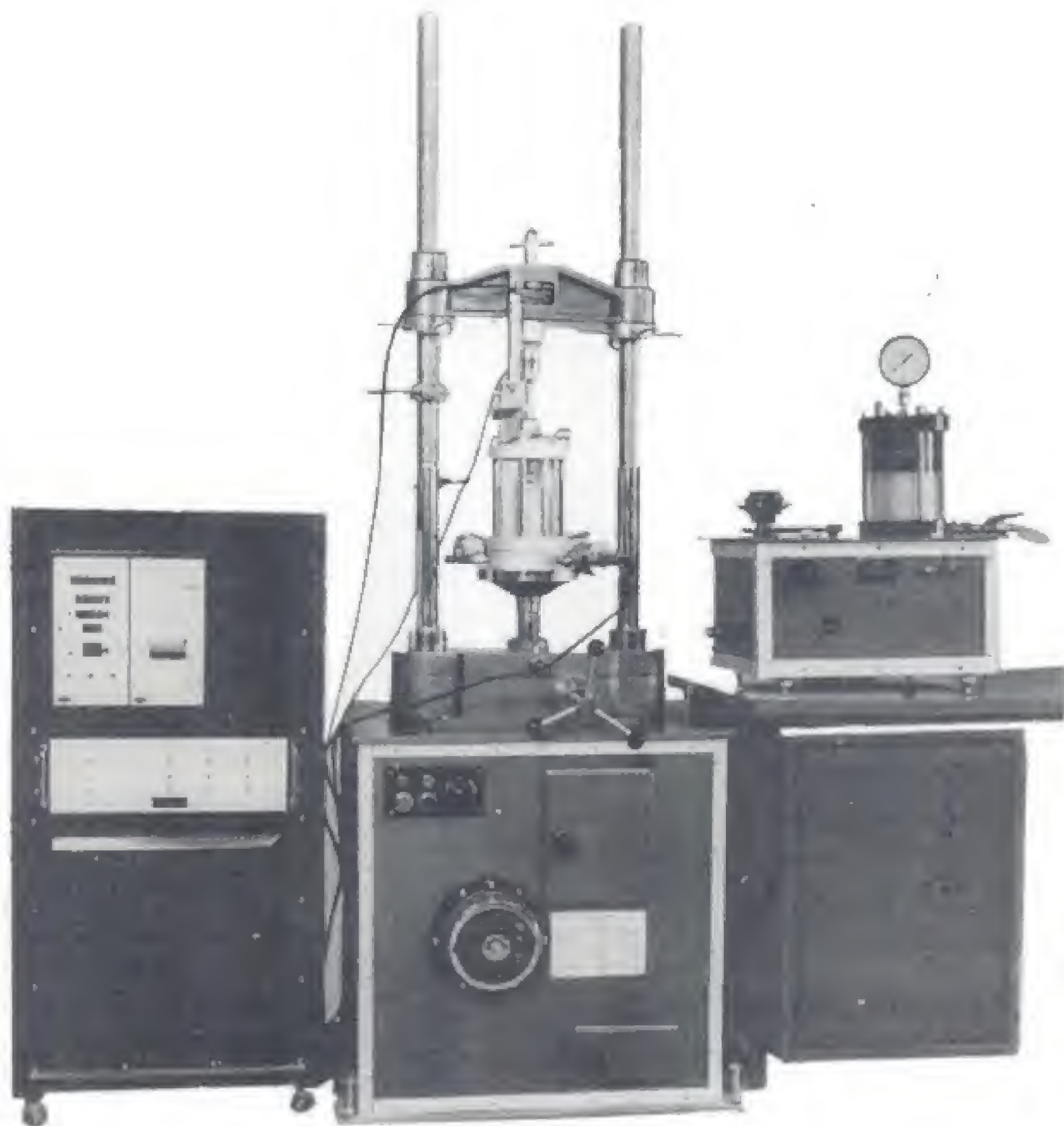
### 33.9. HYDRAULICALLY OPERATED CONSOLIDATION CELLS

Plate XVIII shows the photograph of a hydraulically operated consolidation cell, manufactured by M/s GDS Instruments Ltd., England. The GDS hydraulically operated consolidation cell are available for test specimens of 50, 63.5, 70, 76.2 and 100 mm diameter. The cells are based on the design of Rowe & Barden's hydraulic oedometer. The GDS cells incorporate Bishop & Skinner's novel "floating" ring. Axial total stress is applied by a water-filled pressurised rubber membrane, in contact with the top of the test specimen. This, membrane, which is a perfect fit in the top chamber is not captive but is free to move with axial deformation during soil consolidation or swelling.

Back pressure may be applied to the top of the test specimen via a tube passing through the top of the cell and fixed to the underside of the membrane. Accordingly, large variations in vertical effective stress and permeability may be avoided and field hydraulic gradients properly modelled.

Pore pressure may be measured at the base of the test specimen. GDS has standardised on Legris push-fit connectors for ease of setting up. The cells do not use dead-weights, a loading frame or compression machine and are bench mounted. The cells are each provided



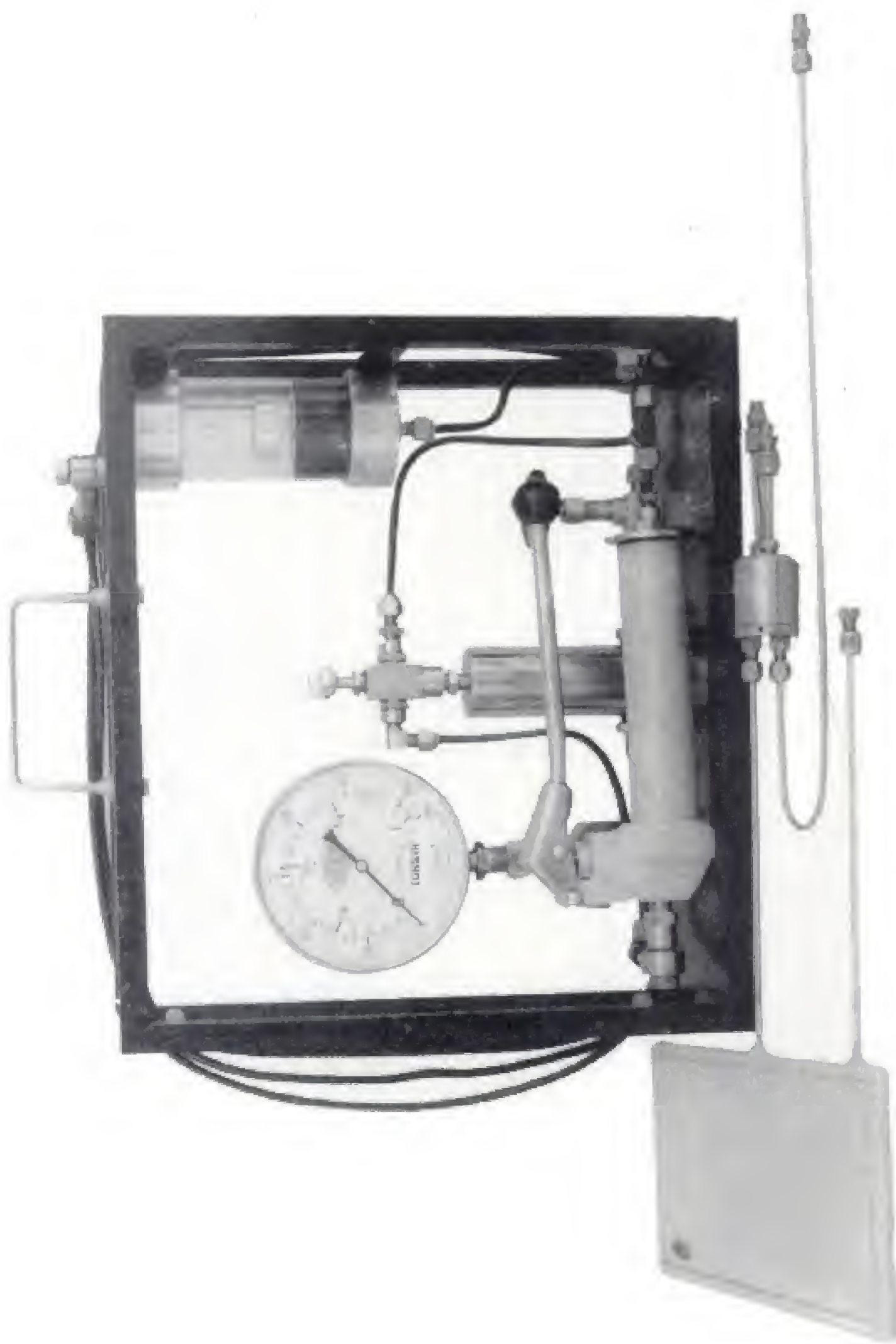


**ELECTRONIC TRIAXIAL TEST APPARATUS WITH DATA LOGGER (HEICO)**

**PLATE XIII**

(To Face Page 880)

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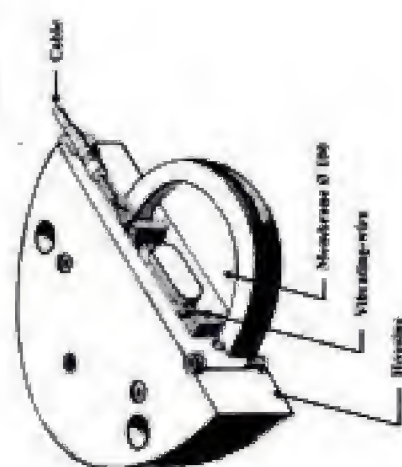


HYDRAULIC EARTH PRESSURE CELL (HEICO)

PLATE XIV



(a) VIBRATING WIRE EARTH PRESSURE CELL WITH CABLE AND READOUT UNIT



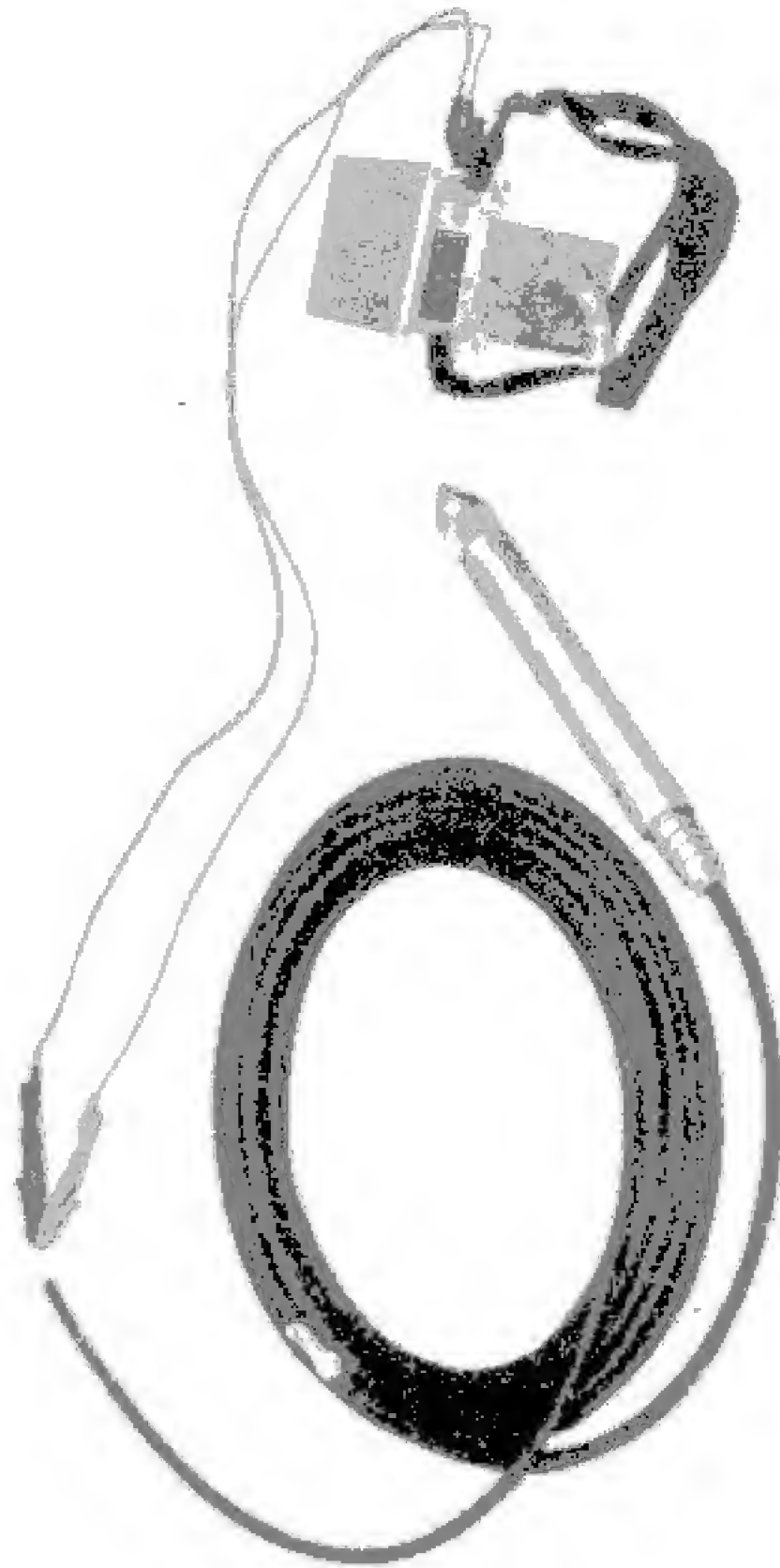
(b) CELL INTERIOR



(c) P105 CELL IN EARTH EMBANKMENT

(AIMIL/GEONAR)  
PLATE XV





VIBRATING WIRE PIEZOMETER WITH CABLE AND READOUT UNIT  
(AIMIL/GEONAR)

PLATE XVI



THE GDS CONSOLIDATION CELL (HYDRAULICALLY OPERATED)



THE GDS TRIAXIAL CELL 100

PLATE XX



time deformation data from multiple test stations. A test station comprises a conventional one-dimensional consolidation apparatus of the Bishop or Casagrande type with dead weight/lever arm front or rear loading.

In addition to logging time-deformation data to disk, the system automatically detects step loading changes and computes coefficient of consolidation for each load increment.

For the measurement of deformation, the system uses the new technology digital indicator shown in plate XVII. The digital indicator looks similar to a dial gauge. Inside the indicator, there is an opto-electronic encoder which gauges the displacement of a finely graduated glass plate. The indicator is provided with a digital output socket.

The advantages of this type of displacement transducer over conventional instruments of the resistive, inductive and capacitive type are as follows :

(i) the indicator is an inherently digital device and not subject to thermal and electronic drift.

(ii) the digital indicator displays its own reading.

(iii) the digital indicator does not require any calibration whatsoever.

The resolution of the digital indicator is 0.001 mm (1  $\mu$ m) and the range is 12.7 mm.

The system is under computer control. Any test may start or finish at any time. The digital indicators are connected by 5 m cables to computer interface units which take up to 8 indicators each. These interface units can be stacked together giving a total capacity of 80 test stations. The interface units are linked to a desktop computer via the interface bus and cables.

From the computer keyboard, the user can define the test station set-up and the data-logging rate. Data can be saved at regular intervals with respect to time, square root of time, logarithm of time. Data can also be stored, perhaps more logically, when a given test station reading has changed by a specified amount. The default is to save the readings whenever they have changed. This has the advantage of concentrating data where they are most needed—where rapid and large changes are occurring, while avoiding filling up disks with data that are changing little.

The system automatically detects step loading changes and identifies the deformations immediately prior to the step loading changes. Before plotting or tabulating saved data, the user has the option to enter the value of axial total stress in any units at each step loading change. From these data, the system synthesises a look-up table representing the load-deformation relationship. From user-defined final voids ratio, the system computes intermediate voids ratios and plots  $e$ -log  $p$  curve.

The system carries out real time logging of time-deformation data to disk. Saved data may be tabulated or plotted or read directly into a users interpretive program. Data may be saved in a format suitable for transmission to other computers.

Saved data may be plotted in various forms. Any one of eight test parameters can be plotted against another. The test parameters are : time, square root of time, logarithm of time, axial deformation, percent axial strain, axial total stress, logarithm of axial total

stress, voids ratio. In addition, the system automatically computes the coefficient of consolidation for each load increment.

### 33.9. HYDRAULICALLY OPERATED CONSOLIDATION CELLS

Plate XVIII shows the photograph of a hydraulically operated consolidation cell, manufactured by M/s GDS Instruments Ltd., England. The GDS hydraulically operated consolidation cell are available for test specimens of 50, 63.5, 70, 76.2 and 100 mm diameter. The cells are based on the design of Rowe & Barden's hydraulic oedometer. The GDS cells incorporate Bishop & Skinner's novel "floating" ring. Axial total stress is applied by a water-filled pressurised rubber membrane, in contact with the top of the test specimen. This, membrane, which is a perfect fit in the top chamber is not captive but is free to move with axial deformation during soil consolidation or swelling.

Back pressure may be applied to the top of the test specimen via a tube passing through the top of the cell and fixed to the underside of the membrane. Accordingly, large variations in vertical effective stress and permeability may be avoided and field hydraulic gradients properly modelled.

Pore pressure may be measured at the base of the test specimen. GDS has standardised on Legris push-fit connectors for ease of setting up. The cells do not use dead-weights, a loading frame or compression machine and are bench mounted. The cells are each provided with a cutting shoe, dial gauge and flexible and rigid drainage discs.

#### The GDS Consolidation Testing System

The GDS consolidation testing system provides desk-top computer automation of test control, data logging, data reduction into SI units, on line screen graphics, and presentation of saved data by tabulation or by colour graphics plotter for direct inclusion in reports.

The hydraulic Rowe & Barden-type consolidation cell incorporates Bishop & Skinner's novel floating ring. The cell is available in a range of sizes of test specimens of 50, 63.5, 70, 76.2 and 100 mm diameter. Back pressure is applied to the cell so unrealistic and complex variations in vertical effective stress and permeability can be eliminated and field hydraulic gradients properly modelled.

Average axial stress is computed from the pressure applied to the top membrane by the axial stress controller. Accordingly, load cells are eliminated.

The test cell is linked to the dedicated controlling computer by three GDS digital controllers. Axial stress, back pressure and pore pressure are each controlled by a 200 cc/2000 kPa GDS digital controller.

Axial deformation is computed from the volume change of the back pressure controller or optionally from corrected volume change of the axial stress controller. Displacement transducers are therefore eliminated.

The system is user-friendly with prompts, questions and information automatically appearing on the computer screen to guide the operator every step of the way.

Tests are chosen from the following test menu :



The GDS consolidation testing system may become the GDS triaxial testing system 38 by simply changing the test cell and software package.

### **33.10. THE GDS 1000 cc/2000 kPa DIGITAL CONTROLLER**

The GDS 1000 cc/2000 kPa digital controller is a microprocessor controlled hydraulic actuator for the precise regulation and measurement of liquid pressure and liquid volume change. (Plate XIX). The volumetric capacity is 1000 cc and the pressure range is 0-2000 kPa.

Deaerated water in a cylinder is pressurised and displaced by a piston moving in the cylinder. The piston is actuated by a ball screw turned in a captive ball nut by a stepping motor and gearbox that move rectilinearly on a ball slide. Pressure is detected by means of an integral solid state pressure transducer. Control algorithms are built into the programmable memory to cause the controller to seek to a target pressure or step to a target volume change. Volume change is detected by counting the steps of the stepping motor. In stand alone mode, the instrument is a general purpose constant pressure source, a volume change gauge, a pore pressure measuring system, a flow pump and a digital pipette. As a constant pressure source, it replaces mercury column, compressed air, pumped oil and dead weight devices.

It can be programmed through its own control panel to ramp and cycle pressure and volume change linearly with respect to time. In computer control mode, it is a full-fledged computer peripheral enabling computer-automated test control and data logging via the standard IEEE-488 computer interface. The User interface is a control panel comprising a 40 character liquid crystal display (LCD) and a 16 key membrane touch panel keypad with audio feedback.

The display is divided into three zones : to show a continually updated display of the current pressure in kPa ; to prompt for information and to display entered information ; to show a continually updated display of volume change since power-on or since the volume reading was last zeroed.

The 16 key keypad is used for entering target pressure, target volume, ramping data and other complementary functions. The keypad also gives access to on-board diagnostics for checking out each of the major hardware components of the system.

### **33.11. HYDRAULICALLY OPERATED TRIAXIAL CELL**

Plate XX shows a GDS triaxial compression/ extension cell 100, which is a hydraulically actuated triaxial compression/extension cell for test specimens of 50, 70 and 100 mm diameter. The title cover of the book shows the photograph of GDS cell 38 for test specimens of 38 mm diameter. The cell is based on the design of Bishop & Wesley's hydraulic triaxial apparatus for controlled stress path testing developed at Imperial College of Science & Technology, London. It may be noted that any test, including a conventional test may be referred to as a "stress path test".

Although Bishop & Wesley set out design is a simple form of triaxial apparatus in which the stress paths encountered in engineering practice can be approximated to more readily than in conventional equipment, in a GDS computer controlled system their versatile cell is equally adept at carrying out classic "standard" tests as well as advanced tests.

Axial force is exerted on the test specimen by means of a piston fixed to the base pedestal. This piston moves vertically up and down in a "Rotolin" linear guide housed in a turret joining the cell to the base. This piston is actuated hydraulically from an integral lower chamber in the base of the cell which contains pressurised deaerated water. The piston is sealed into the cell and into the lower chamber by matched Bellofram Rolling Diaphragms which sweep equal volume of water. Cell pressure, back pressure, and pore pressure connections are made in the usual way except that GDS has standardised on Legris push-fit connectors for ease of setting-up.

In the GDS triaxial systems, an axial load cell is not used. The computer uses Bishop & Wesley's equations to compute axial stress from the cell and lower chamber pressure and from the current cross-sectional area.

In place of the load cell, the GDS extension device is fitted as standard. It is simple to set up and allows smooth transitions between compression and extension. A surgical vylastic sleeve seals a plane top cap to a fixed reaction head and prevents cell pressure from acting vertically on the test specimen. Accordingly, axial stress may be reduced below cell pressure.

The cell is supplied with interchangeable base pedestals and top caps accommodating 50, 70 and 100 mm diameter test specimens.

The cell does not use dead weights, a loading frame or compression machine and is bench-mounted.

### The GDS Triaxial Testing System 38

The GDS triaxial testing system 38 provides desk-top computer automation of triaxial test control, data logging, data reduction into SI units, on-line screen graphics, and presentation of saved data by tabulation or by colour graphics plotter for direct inclusion in reports (Fig. 33.12)

The hydraulic Bishop & Wesley-type triaxial compression/extension cell tests 38 mm diameter test specimens. The test cell is linked to the dedicated controlling computer by three GDS digital controllers. Axial stress, radial stress and back pressure/pore pressure

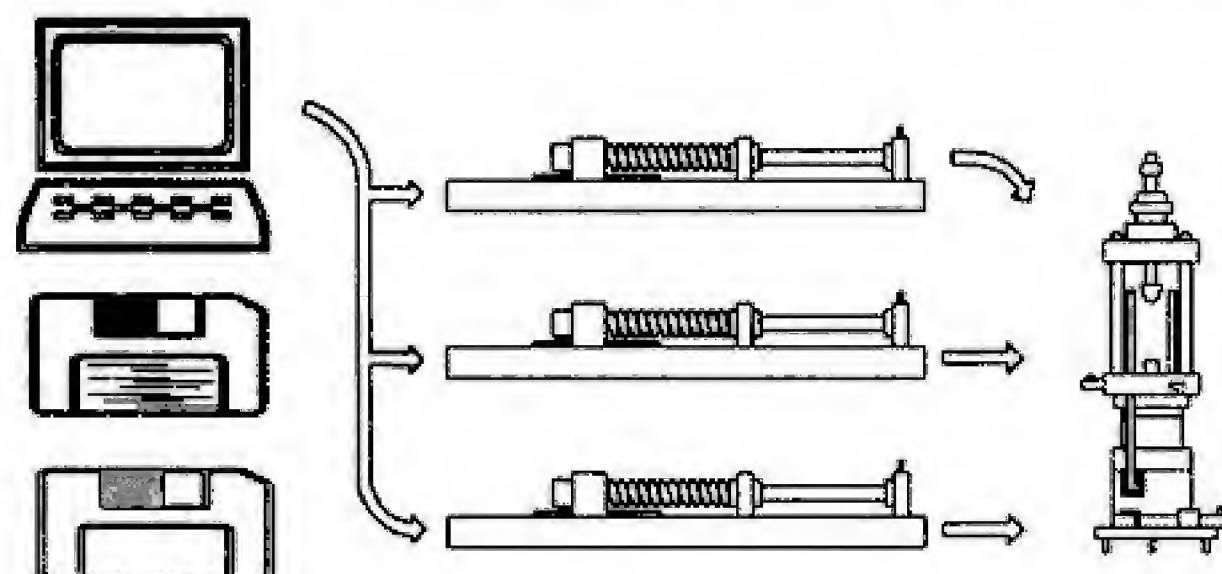


FIG. 33.12. THE GDS TRIAXIAL TESTING SYSTEM 38.



are each controlled by a 200 cc/2000 kPa GDS digital controller. Using Bishop & Wesley's equations, the system continuously computes axial stress by using the pressures measured by the axial and radial stress controllers. The GDS extension device is a standard fitting in place of the redundant load cell.

Axial deformation is computed from the volume change in the axial stress controller. Accordingly, displacement transducers are eliminated.

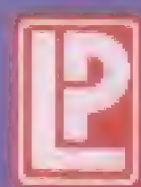
The system is user-friendly with prompts, questions and information automatically appearing on the computer screen to guide the operator every step of the way.

Tests are chosen from the following test menu :

1. Isotropic consolidation and swelling
2. Evaluation of pore pressure parameter  $B$
3. Saturation by simultaneous ramping of back pressure and pore pressure
4. Unconsolidated-undrained triaxial compression/extension
5. Consolidated-undrained triaxial compression/extension with measurement of pore pressure.
6. Drained triaxial compression/extension with measurement of volume change
7. K-zero consolidation and swelling to an over-consolidation ratio, drained and undrained continuous linear stress paths with smooth transitions between compression and extension.
8. Cyclic loading by sinusoidal, triangular and square wave forms ; periods down to sea wave periods.,
9. Permeability by constant rate of flow or by controlled hydraulic gradient.

Using the system's "loop-round" facility, any number and combination of the tests may be sequentially carried out on the same test specimen. The current height and diameter of the test specimen are passed on from the end of one test to the beginning of the next. Tests may run unattended and continuously for any period of time.

Tests are optionally axial stress controlled or axial deformation controlled. There is no minimum testing rate. The value of the average cross-sectional area used in all computations is continuously corrected for axial deformation and for volume change. Saved data may be corrected for the effect of membrane stiffness. Automatic scaling of plotted data may be over-ridden to allow details to be enlarged or to allow families of curves to be plotted within the same set of axes. The GDS triaxial testing system 38 may become the GDS consolidation testing system by simply changing the test cell and software package.



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